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STANDARDIZATION OF HYDRANT MARKINGS.

BY SIDNEY S. ANTHONY.*

[*Read February 8, 1933.*]

For many years attempts have been made by water and fire departments all over the country to indicate by means of colors the various capacities of fire hydrants. No two cities have adopted the same scheme; consequently it has been difficult for fire departments called in from the outside in case of large fires to determine what to expect from a hydrant to which they may have been assigned or which they picked themselves.

Usually, of course, assisting departments are guided by local officials, but frequently they are not. If not, they may connect to the nearest hydrant which may have been left vacant, because the local department knew that it was not of sufficient capacity. Then they complain of lack of water pressure and blame the local water department.

In other cases, the local department itself may not know that one hydrant is better than another. Frequently adjacent hydrants are fed by mains of different sizes, one on this corner from a 6-in line and one on the next corner by a 16-in. line. We have cases like this at Augusta, and every city or town must have similar arrangements.

The matter of standardizing hydrant markings was brought to the attention of the Maine Water Utilities Association formally in June, 1928, in a paper read by James W. Graham, former General Manager of the Portland Water District. In his paper, Mr. Graham reminded us that the Underwriters' requirements for pumpers are based upon a discharge pressure of 120 lb. per sq. in. and that where long lines of hose are used, it is necessary to carry higher discharge pressures which result in reducing the rated capacity of pumpers. A pump will deliver only one-half its normal discharge at 200 lb. per sq. in. discharge pressure and only one-third at 250 lb. per sq. in. If a residual pressure of 100 lb. per sq. in. could be maintained, the pumper would deliver its rated capacity at 220 lb. per sq. in., but if the suction pressure is reduced to zero the pump can deliver only half its capacity.

It is important that the influence of size of main and pressure be understood by the fire chief so that mains may not be overloaded with apparatus.

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In small towns, where volunteer departments are the rule, a hydrant is a hydrant, a place to obtain practically unlimited quantities of water for the extinguishment of fires. If there is not enough water, the water department immediately hears about it, even if the hydrant is at the end of a mile or two of 6-in. pipe. Static pressure in a case like this means little, particularly if the line is old and partially filled with rust.

In every fire station there should be a map of the distribution system showing the size of mains and location of hydrants. The capacity of each hydrant in gallons per minute could also be noted upon it. The firemen should be required to study this map and know what to expect from each hydrant. This will not assist the visiting firemen, however. Also, when a building is aflame possibly with people in it crying for help, the average man will forget his fire station "skull practice" and think only of getting water on the fire and ladders to the windows.

Many cities have a high-pressure service and a low-pressure service. Hydrants are usually marked to tell with which system they are connected.

That does not mean that all the high-pressure hydrants have plenty of water in them, however, for a high-pressure hydrant may be on the end of a long line, dead ended, and near the top of the hill, and may be able to supply less water than a low-pressure hydrant lower down in the city or town.

A committee of the Maine Association looked into the matter of hydrant marking, the writer being a member of that committee. We held several meetings, conferred with officials of the Insurance Exchange and similar organizations, discussed the subject in open meeting, and finally brought in a set of rules which we recommended for adoption by the Maine Water Utilities Association.

These rules are substantially as follows:

The committee are unanimous in their opinion that a standard system of grading of fire hydrants as to their value to the fire department should be adopted by this Association.

The majority of the committee favor three groups of hydrants to be designed as Class A, Class B, and Class C.

Class A group should be fire hydrants which have sufficient capacity to supply one or more pumpers with 1 000 g.p.m. or more with a residual pressure of not less than 10 lb. per sq. in.

Class B group should be fire hydrants which have sufficient capacity to supply pumpers with not less than 500 g.p.m. with a residual pressure of not less than 10 lb. per sq. in.

Class C group should be fire hydrants which have insufficient capacity to supply pumpers with 500 g.p.m. with a residual pressure of not less than 10 lb. per sq. in.

Class A group should be designated with *green* tops and nozzle caps.

Class B group should be designated with *orange* tops and nozzle caps.

Class C group should be designated with *red* tops and nozzle caps.

In order to determine the value of the fire hydrants in the different groups, it would be necessary for the water department to carry out a flow test inasmuch as the size and the static pressure of the main to which the

hydrant may be connected have very little value unless the main is located on the gridiron system.

The committee are also of the opinion that the matter of pressure is of secondary consideration inasmuch as it is the duty of the water department to furnish water in sufficient quantity at ordinary pressures which is ample for good domestic service, and that all higher pressures required should be provided by the fire department by the use of pumpers or fire engines.

The committee also recommended that all hydrant barrels be painted orange for maximum visibility in all conditions of weather and light, this being found after experiments by members of the committee to be the best color.

This report occasioned a great deal of discussion, not only in Maine, but all over the country. The regulations were published in several professional magazines and have been quoted by them in reply to questions from fire chiefs and water superintendents in various sections of the country.

Early this year the committee was instructed to submit the rules to the American Water Works Association, the New England Water Works Association, and the New England Fire Chiefs Association, and recommend that the rules or similar regulations be adopted and made uniform throughout the country. This was done, and at least one of these, the New England Fire Chiefs Association, has appointed a committee to go over the rules and bring in a report, which I understand will be done at its June convention this year.

In Maine, many water utilities have adopted these regulations and graded and marked their hydrants accordingly. Most of the Kennebec Valley is so marked, and during the past summer, the Portland Water District engaged in perhaps the most elaborate classification of hydrants of any of the member companies, and probably also the most accurate one as to results.

In Augusta, the summer following the adoption of the regulations, we tested our hydrants and painted them their proper colors. We did not, however, test every hydrant, merely taking those on the outskirts, grading them green, orange or red, as the case may be, and working in toward the center of the system until we found a green-top hydrant. Then we painted all hydrants inside those limits green, and called the work good.

In some cases we had to come in quite a way to get a green top. One line, in particular, has 18 red top hydrants on it, the line being five miles long, dead ended, and without a side feeder. The three hydrants nearest the supply end are green, the next two, yellow, and the remaining eighteen red, clear to the town line and the legal limit of the District. The fire department has never been able to get more than one satisfactory fire stream from these hydrants, even with a pumper, as a vacuum is created, even without the pumper, on the last few. We are now engaged in laying a large feeder nearly half way out to this dead end. The work will take about three years more to finish. Then with a standpipe midway of the remainder,

we may get some decent fire protection in that end of the city. Some of our neighbors have conditions nearly as bad as that, but I guess there is no one to equal us.

The system of grading has been commended highly by the Chief of the Augusta Fire Department and by many others in the state of Maine. Chief McCurdy of Augusta states that now, when his men find they are not getting as much water as they desire from a green-top hydrant, they investigate to see if they have the hydrant wide open, a kink in the suction hose, or other trouble, before blaming the Water District for lack of pressure. When they are connected to an orange- or red-top hydrant they use particular care to see that they do not create a vacuum, which causes trouble in house plumbing or sucks air in and breaks up the effect of the fire stream as bubbles of air in a hose tip will reduce considerably the effectiveness of the stream.

HYDRANT PRESSURE AND FLOWAGE TESTS AT PORTLAND, MAINE.

BY LEAVITT R. SMITH.*

[*Read February 8, 1933.*]

After some discussion on standard markings for fire hydrants at the June, 1928, meeting of the Maine Water Utilities Association, President David E. Moulton appointed a committee to study this subject. After several committee meetings with much careful study, and with the approval of the chief engineers of the various fire departments the committee reported.

In order to comply with the recommendations of the Maine Water Utilities Association† for the marking of hydrants, the Portland Water District during the summer of 1932 conducted tests on 750 or 41 per cent. of the hydrants in its system. These hydrants are situated in ten of the twelve cities and towns to which it supplies water. Each hydrant was tested individually. The static and residual pressures were taken and a flow test made, using a Pitot tube held in the hydrant stream. It is expected that the remaining 59 per cent. of the District's hydrants will be tested in the summer of 1933.

Record of Tests. In the Water District's office is filed a card for each hydrant giving its location and other information. These cards were numbered, starting with the number 1 for each city or town and marking the cards consecutively. Plans were prepared showing the location of hydrants tested and the gates and mains. On these plans the hydrants were marked with the number corresponding to that on the cards. The plans were drawn on a scale of 500 ft. to the inch, and this scale proved very satisfactory for field use. A field book was prepared with proper ruling of the pages in columns, and the hydrant numbers were inserted in this book so that the information obtained as each hydrant was tested could be easily recorded.

Testing Party and Equipment. The party performing the tests consisted of from two to four men, depending on how hard it was to keep catch basins free, fill in wash outs and prevent the water from damaging property. The party used two tested pressure gages registering from 1 to 100 lb. per sq. in., a hydrant wrench, a gate wrench, a Stillson wrench, two large pieces of canvas 8 ft. by 12 ft. in size, and three 1¼ in. by 4 ft. iron rods. The canvas was used to keep the water from washing out esplanades and lawns.

Method of Testing. One of the gages was connected to the hydrant

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†That a uniform color marking of fire hydrants should be used throughout the state. That the colors be those used on traffic lights; green, yellow and red. Green to signify a capacity of 1 000 g.p.m. or more, yellow, from 500 to 1 000 g.p.m. and caution should be used, red, less than 500 g.p.m., and care must be exercised not to create a vacuum by breaking the stream with a pumper.

With this system of colors to guide them, fire departments called from one city or town to another would know at a glance the capacity of any hydrant they may wish to use.

outlet and the other to the hydrant Pitot tube. When the hydrants were opened wide, the Pitot gage usually recorded about 2 lb. per sq. in. less pressure than the pressure gage, as a result of inserting the Pitot blade into the stream and backing up the flow slightly. We have learned that New York and some other places have discontinued the use of hydrant Pitot tubes and rely on pressure gages only for results.

The Pitot blades cost $3.00 each. When rocks or other foreign matters strike them they are ruined. As many as three blades were broken in testing one hydrant, but one blade will test on an average about 50 hydrants. Since the residual pressure equals that registered on the Pitot gage, we shall discontinue the use of the Pitot tube when we resume our tests.

On long lines of pipe leading to dead ends, we found it advisable to start testing at the end nearest the supply, thus keeping dirty water that may result from the stirring action of the abnormal rates of flow moving along with the party. We also found it good practice to locate and test each gate on hydrant branches before making the hydrant test, thus assuring the possibility of a shut down in case something became lodged in the hydrant valve or some defect was found in the mechanism of the hydrant. After finding that the gate could be used, the hydrant caps were removed and enough water discharged to wash out any sediment lodged in the parts. Only then was the pressure gage attached.

If the hydrant jet struck soft ground, canvas was spread with one end tied to the hydrant. If it became necessary to deflect the water, one edge of the canvas was tied to the tops of the iron rods driven into the ground, in line, and diagonal to the flow.

Flushing. The hydrants were opened wide and flushed until all signs of sediment had disappeared; then the Pitot blade was inserted in the stream and the reading on both gages taken and entered in the note-book opposite the hydrant number shown on the plan. Any defect in the operation of the hydrant was also noted. It is necessary to open the hydrant wide in order that the pressure of the hydrant may not be reduced by a partially open hydrant valve.

At some places it was unwise to test the hydrant because of the possibility of property damage. In such cases the static pressure was taken in order to aid in figuring the flow in the office. It is of assistance if the hydrants on each side of the one in question have been tested.

The requirements were that the delivery of the hydrants at a residual pressure of 10 lb. per sq. in. was to be ascertained. However, a reading of exactly 10 lb. per sq. in. was seldom registered with the hydrant opened wide. By the use of the hydraulic slide rule it was possible to figure what the flow would be at 10 lb. per sq. in. after finding what it was at the pressure actually recorded. The company manufacturing Pitot gages supplies charts reading in equivalent g.p.m. for each lb. per sq. in. registered on the gage from 1 to 36 lb. per sq. in. inclusive. We prepared a curve showing all values we had to use that were not listed on this chart.

Weak Places in Mains. During our tests three breaks occurred in the system. One was found to be due to a thin place in a 6-in. pipe; the second was due to an old crack in a 6-in. hydrant tee; the third was due to an old crack in a 6-in. pipe. It appeared that these places had been waiting for the extra pressure caused by closing the hydrants to make them break.

Results. A tabulation of our tests showed very interesting variations in different parts of the system. Of the hydrants in South Portland 86 per cent. were green, while 100 per cent. of those in West Falmouth were red. These are the two extreme cases. Taken as a whole, one-half of the hydrants are green, one-third yellow and the rest red.

A summary of findings for the districts completed is given below.

	Number of Hydrants Colored.			Total.	Per Cent. of Hydrants Colored.		
City or Town.	Green.	Yellow.	Red.		Green.	Yellow.	Red.
South Portland	197	27	4	228	86.5	11.8	1.7
Westbrook	106	30	28	164	64.7	18.3	17.0
Scarboro	19	58	12	89	21.5	65.0	13.5
Cape Elizabeth	24	53	12	89	27.5	59.0	13.5
Falmouth	50	12	0	62	70.6	19.4	0.0
Gorham	1	27	15	43	2.0	63.0	35.0
Cumberland	4	7	23	34	12.0	20.4	67.6
West Falmouth	0	0	23	23	0.0	0.0	100.0
South Windham	1	9	3	13	8.0	69.0	23.0
Great Diamond Island	3	2	0	5	60.0	40.0	0.0
Total	406	225	119	750	54.1	30.0	15.9

Ownership of Hydrants and Hydrant Defects. For a period of years after a hydrant is set, the municipality served by it pays a rental fee to the Portland Water District. After this term of years expires, the hydrant becomes the property of the municipality which is then responsible for its maintenance. When the water utility conducts hydrant tests, mechanical defects are often discovered in localities where the municipal officials do not properly inspect hydrants.

In one village with a total of 13 hydrants, 7 were controlled by the municipal officials, and the others by the Water District. The caps on 5 of these 7 hydrants had to be hammered to remove them. Two men on a wrench could not open these hydrants and a 10-pound hammer had to be used on the wrench to start water flowing. From 10 to 15 minutes were required to open these hydrants.

In a city adjacent to Portland between 25 and 30 gate boxes on hydrant branches controlled by the city were found to be filled with sticks and stones. After a couple of months an order was passed by the city council

asking the Water District to clean out these boxes; only then could the test be completed.

Among the mechanical parts of the hydrants found out of order were the following: Caps missing, 1; ports to be leaded in, 2; loose valves, 19; stuffing box leaks, 10; valve gaskets leaks, 8; gate boxes filled, 45; top sections of gate boxes gone or tipped, 4; gate boxes under concrete road, 4; gates repaired, 1. Out of 750 hydrants tested, 94 or about 13 per cent. had some defect. These figures indicate that the mechanical parts of our hydrants should be tested more frequently.

Conclusion. The results obtained in this study are believed to be valuable and well worth the expenditure incurred.

The keeping of good records and assembling of necessary apparatus before starting the field work are essential.

The location of each hydrant gate should be known before starting the test.

Testing should begin nearest the source of supply and move toward dead ends, and every hydrant should be flushed until it delivers clear clean water.

Use of the hydrant Pitot gage is unnecessary, as the residual pressure gage gives the same results. After tests are completed the water utilities and fire departments know that every hydrant and gate is in good working condition, and that all defects have been corrected.

The amount of water available at each hydrant is indicated by its color and both the local and foreign apparatus can work to best advantage.

INSPECTION AND MAINTENANCE OF GATE VALVES.

BY C. W. MOWRY.*

[*Read February 8, 1933.*]

So far as I can discover from the Index of Transactions of this Association, it is nearly 20 years since a paper was presented on the subject of inspection of gate valves. In 1914, Patrick Gear of Holyoke read a paper on "Care of Gates and Hydrants," in which he stressed the need of regular inspections and careful maintenance. In discussing Gear's paper, J. M. Diven, a past president of the American Water Works Association, said that he was once put in charge of a water works that had neglected valve tests, and on making an inspection and test of valves, found 244 out of about 1 100 valves in some way defective; that is, with broken stems, obstructed boxes, or in need of packing. In other words, about 22 per cent. of the gates inspected were found defective.

Conditions as bad as those described by Diven, probably do not exist today, but what may be found if valves are not regularly inspected is well illustrated by the recent and interesting experience of the Philadelphia Water Works. At the suggestion of Distribution Engineer Levering, an inspection of all the valves in public mains in the city was started in 1926. For this work, three crews were organized, and approximately two years were required for completing the inspection. Not all valves were examined on the first round, because if a valve box was not readily located or was found filled with dirt or stones it was passed. This first partial inspection revealed 1 150 valves or 4 per cent. improperly closed, as well as a large number of others that were inoperative due to stems being stripped or other causes. The total number found in operating condition was about 28 000.

The second inspection, following about two years later, extended over another two years. It was more thorough than the first and revealed 38 000 valves in operating condition. Although a large proportion of the defects found during the first inspection had been corrected, yet 950 valves were found improperly closed, as compared with 1 150 on the first inspection. A correspondingly reduced number of valves was inoperative.

A third inspection is now under way, but progress is slower on account of a reduction in the force of men. So far, 18 000 valves have been found in operative condition, and 167 have been found to be closed. Minor defects are decreasing.

This account of the Philadelphia experience is not intended to be a complete report, but is an excellent example of a fine piece of work, and demonstrates the need of regular inspections.

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We have just received a report from one of our field men regarding the practice of Cleveland, Ohio, with respect to this subject. When a foreman closes valves for repair work, he lists them on his work sheet and is instructed to check the list when the valves are reopened. In the main office there is kept a permanent record of all these valve operations, compiled from the foreman's work sheets. Once a year, the Cleveland water department inspects all valves in the distribution system regardless of size, including service connections and hydrants. All valves are operated the full travel, down and up, counting the number of turns in both directions. Of the 6 000 valves inspected since August of last year, 20 were found closed, demonstrating the need of regular valve inspections in spite of the precautions taken by the foremen.

The value of gate-valve inspection and maintenance is three-fold. First, inspections are necessary to see if all valves are readily accessible for emergency use. Valve boxes, if covered by paving or earth, can be located and re-set properly; if dirt and stones are found in the valve boxes, the obstructions can be removed.

A second value of regular valve inspections, is to determine if the valves, although accessible, are in operative condition. Valves that have been left in one position for years, become corroded, so that they cannot be closed, or that so much effort must be applied to move the gate mechanism that parts are broken. By pouring a small quantity of kerosene or diluted lubricating oil down the valve-key, some of the oil will run over the valve-operating nut and lubricate the joint between the valve stem and packing. Sometimes valve stems can be turned indefinitely, indicating a broken stem.

Another trouble is that sediment accumulates in the valve seat and prevents closing the valve tightly. By carefully operating the gate up and down and allowing water at high velocity to pass through the nearly-closed gate, the groove can usually be cleaned sufficiently to allow the gate to seat properly. Some valves start to leak badly around the stem when operated, making it necessary to uncover the valve and re-pack the gland, which is an expensive job. It has been suggested that on well-paved and important streets, it would be economical in the long run to install valves in brick or concrete pits, large enough to permit packing the valves.

In this connection, W. E. MacDonald, Water Works Engineer of Ottawa, in a recent paper on the operation of a Canadian water works system, stated that in that particular system all gate valves are installed in valve pits and are inspected at least annually for the detection of gland leakage and to ascertain if the valves are in proper working order. He stated that at such inspections the stuffing boxes are packed, the gearing is lubricated, and a test is made to determine that the valve is fully open.

A third value of regular valve inspections, in addition to seeing that valves are both accessible and operative, is to determine that the valves are open. In a discussion of the paper by Gear, previously referred to, P. R. Sanders cited an experience with a large, geared gate valve. The men had

put the key on the small gear and turned it for a while. Then, finding that the key turned easily, they thought the valve was about half open and, shifting the key to the big gear, they continued to turn it in the same direction, but instead of opening the gate they shut it down again.

A closed gate in Peabody, Massachusetts, was revealed three months ago, by a fire, when several sprinklered buildings of a tannery burned with a loss of about $200 000. An investigation was made to see why the automatic sprinkler system had not controlled the fire at the start. The only water supply to the sprinklers was through a connection from the city main, and the gate valve in the street on this connection was found shut. A few weeks previous to the fire, a flow test had been made from a 2-in. drain on the sprinkler system, and no lack of pressure was observed. The water department had not operated the valve on the fire-service connection for years, and it has not yet been discovered who closed the valve, or why it was closed.

In private fire protection systems, great care is exercised to keep sprinkler valves open. It is standard practice for the owners to seal sprinkler valves in the open position, and to inspect the valves weekly. In addition to this, insurance inspectors check the valves at regular intervals of about three months and make water-flow tests during the summer to prove still further that the valves are open.

In spite of all these precautions, inspectors of the Factory Mutual Insurance Companies during the year 1932, found 219 improperly closed sprinkler valves, of which nine were on connections from city mains. In 1931 they found 195 improperly closed valves, of which nine also were on connections from city mains. In some cases, as was true in Peabody, the valve found closed was on the only water supply to the automatic sprinklers.

At only a relatively few properties, however, can our inspectors examine gate valves in the street during their sprinkler-valve inspections every three months, as these street valves on the fire-service connections are generally considered to be under the supervision of the water department, and the owner does not have a valve wrench.

It would, therefore, be interesting in discussing this subject of inspection of gates in distribution systems, to consider whether or not street valves on private fire-service connections from public mains should always be under the supervision of the water department and included in the regular inspection of gates in the distribution system.

In the *Journal* of the American Water Works Association of September, 1928, page 386, are given the results of a questionnaire in which the following questions were asked:

a. Do you make regular inspections of gate valves?

Replies were received from 64 cities, of which 47, or 74 per cent., answered in the affirmative; and 17 cities, or 26 per cent., answered in the negative.

b. What do you consider an adequate inspection?

Replies were received from 50 cities, of which 24, or 48 per cent., answered that inspections should be yearly; and 12 cities, or 24 per cent., answered that inspections should be semi-annually. The remainder of the cities have regular inspections, but did not state how often they were made.

The New England Water Works Association could well take a leading position in this important matter by recommending to its members the practice of inspecting and testing at least annually, all valves in water-works distribution systems, including the valves on industrial and fire-service connections from public mains. As a specific suggestion to encourage an annual inspection of valves, the Association's standard form for "Summary of Statistics," widely used in the annual reports of water departments, might have an item added calling for the number of valves inspected and tested during the year.

MAINTENANCE OF GATE VALVES IN BOSTON.

BY GEORGE H. FINNERAN.*

[*Read February 8, 1933.*]

In the distribution system of Boston there are about 26 000 gate valves ranging in size from 3 to 36 in. They control main lines, domestic and fire supplies and hydrant and blow-off pipes. Obviously, these gates, to be of value, must be quickly and easily accessible, and in good working order at all times. To assure these requirements, proper operation, frequent inspection and necessary repairs are essential.

As a means for an almost continual check-up, a special crew of two men, a chauffeur and auto truck is engaged in gate inspection and repair work for about nine months in the year. During the remaining three months — December, January and February — their time is given to the care of hydrants, and in salting important gates. This crew works the city in sections, in accordance with 200-ft. scale sectional blue prints of the system, which they carry with them. Every gate indicated on the blue print is located and inspected, and repaired so far as the capacity of the gate crew allows. When repairs beyond their capacity are called for, they are made by other men.

The first move in the gate-crew's routine is to locate the gate. While the blue plan serves as a primary guide, it is not infrequent that the plan needs to be brought into agreement with actual conditions found in the street. In other words, gates may be found which are not on the plan. Furthermore, the plan does not attempt to show all gates on domestic and fire supplies leading into buildings. Data of that class may be had from other sources. It is, therefore, the practice of the inspection crew to scrutinize the street surface for all water fixtures whether they are indicated on the plan or not.

Having located the gate, the crew observes whether or not its location is marked at some easily visible nearby point. Main gates, usually located at street intersections, are marked on the corners of buildings, or on a street sign post, or an electric light standard. Gates in supply lines into buildings are marked on the front of the building supplied. Hydrant gates are marked on the front of the hydrant barrel.

The marks are made with white or black paint, according to the color of the background, and are applied by brush and stencil. They consist of the letter W with additional qualifying letters, such as HS for high service; F for fire pipe; S for domestic supply, etc. Under the letters is placed the distance in feet between the mark and the gate. In the event of the mark not being directly opposite the gate, it is necessary to put an offset distance

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with arrow indicating the direction of the offset. The letters and numbers are about one inch high. They have to be renewed from time to time as they fade out or are painted over. We try to place them on brick or stone structures which are not painted.

The purpose of these marks is to enable us to find the gates quickly when they are covered with snow, ice, road tar, macadam, or when, for one reason or another, they are invisible. The fact that the gas, electric, telephone and other underground public service corporations, have adopted the system for locating their manholes, would seem to indicate that it has some value.

After the location mark is checked, the valve-box cover is observed for proper marking also. All covers are of our own design. They have the letters BWW (Boston Water Works) cast on the surface. In addition they have the letters — FP — SP — HS — BO — DIV. etc.— as the case may be, to indicate that the gate controls a fire pipe, a service pipe, a high service line, a blow off, or a division gate, etc. There are other classifications, but those given are sufficient for explanation.

These cover marks are very helpful to the men, as they give them a picture of the layout underground. Where there are groups of gates at intersections; where high- and low-service mains are closely laid; and at buildings, where several fire and domestic pipes enter; these cover marks are of much assistance. They obviate frequent reference to the plans and tend to make the operation of the gates to produce desired results not so intricate a matter. In the old days, the value of certain employees was rated on their ability to execute complicated shut-downs. They memorized the location of the gates and were more or less secretive as to the effective combinations. That is now a thing of the past.

After checking the cover marking, the cover is removed and the wrench applied to the gate. Incidentally, the interior of the box is observed, whether or not it needs to be cleared of water, mud, or stones. The operating nut is checked — is it securely fixed on the end of the stem — are the corners worn or corroded so that the wrench does not bite? If any of these impediments or defects are observed, they are attended to.

Then comes the most important move. The gate is tried for its condition, whether open or shut. If it is a main gate and found shut, the crew gets in touch with headquarters at once to find out if there is any reason why it should be shut. If there is none, it is opened. If a gate controlling a domestic supply is found closed, it is left closed and reported on return to quarters. If a gate controlling a fire-pipe supply is found down and the building which it enters is occupied, our men usually enter and investigate as to the reason why the fire pipe is shut off. It may be that the occupants are unaware that it is off. It may be that it was shut off for repairs which have not been completed. The men leave it as found and report it upon return to quarters, where the matter is reported at once to the Board of Underwriters, the Fire Department, and the Municipal Building Inspection

Department, all of which get into action to investigate the closed gate. In the meantime, our records are referred to, and very often information is to be had explaining the situation. If there is no good reason why the gate should be closed, it is quickly opened.

A law recently has been enacted in Massachusetts which forbids the closing of a fire-pipe gate without first getting permission from the local fire department. Exception is made in the event of a sudden break in the pipe, when the water department is allowed to shut off the pipe, but a report must quickly be made of its action.

It has been the custom of the Boston department for many years past to notify the Insurance Board, Fire Department and Building Inspection Department, when fire pipes are shut off. It gives the proper parties an opportunity to follow up and have conditions of safety restored.

Hydrant gates are given particular attention with reference to full opening. A partially closed hydrant gate may cause a serious deficiency in flow and pressure during a fire.

Once in a great while a division gate between high and low service, or a blow-off gate, is found open and, of course, quickly closed. This is rated as a prize find. It rarely happens, and we usually do not have to wait for a chance discovery as the condition is indicated on our consumption charts, and we start hot on the trail for the cause.

After determining whether a gate is open or closed, it is run down and up the full range of the screw. The number of turns are counted, and if there is a shortage, the operation up and down is continued, with the probable result that the full range of the screw will be attained. The descending valve breaks up the accumulation of rust in the valve seat and also clears the screw threads of hardened matter that has lodged there.

There are, at times, some gates that we cannot close entirely without pre-arrangement. Large gates in large feed lines, if entirely closed while consumers are up and about, will sometimes reduce the pressure in high buildings and certainly will change the direction of flow with consequent roiling of the water. Hotels, laundries, industrial users and private consumers, will be heard from in short order. It is, therefore, our policy to operate such gates between midnight and morning. Also, the entire closing of a gate in a domestic supply without notification may sometimes cause protest, if not damage or business inconvenience. This has to be considered and provided for.

Incidental to the operation of the gates the packing is often found to leak. Sometimes this can be remedied by turning down the stuffing box or gland bolts with a special wrench. If the packing has lost its body and is shrunken and dry, there is nothing to be done except repack; although stuffing-box leaks may be temporarily stopped by opening up the gate *hard*, or by patiently manipulating it until the relative positions of the stem and packing are such as to stop the leakage. This is only a makeshift, however, and the gate should be repacked. This is where the value of a gate box

sufficiently large to admit a man is appreciated. As most gate boxes are of iron with round bodies from five to seven inches in diameter, digging must be resorted to, at least in northern cities where pipes are laid at depth, and the gate top is not within reach of the street surface.

In the inspection of geared gates our crew also attends to the lubrication. There are one or two places in large geared gates that are equipped with grease cups. These are turned down during inspection. The latest type of geared gate which has the gears encased would seem to be most desirable as the casing can be filled with grease, causing the gears to move easily and avoid rusting. We find it necessary in attending to large gates not only to repack and lubricate, but also to clean the teeth of the gears with wire brushes or compressed air. A large gate should be given the best of care. "He is a very important party."

Before leaving the gate, the grade of the cover with reference to the surrounding pavement is observed and any settlement of either gate box or roadway noted and reported.

A daily report of inspections; repairs, if any; matters to be attended to by others; and information of interest; is made by the crew, and is recorded and cared for in the office. As an addition to inspection, all other employees who have occasion to operate gates are required to observe and report any condition that should receive attention.

It may be of interest to know that our gates open to the right, or clockwise. I have an idea that about half the gates in water systems throughout the country open "right." My experience has been that most men will start to open a gate, or a hydrant, to the left. It seems to be a more natural way. We open a faucet in our homes, to the left. When we wish to open anything that rotates, we turn it to the left. It is, therefore, difficult for a man with that instinct to quickly reverse his mental habit and act in the opposite direction. The consequence, so far as water works are concerned, is a broken or bent gate- or hydrant-stem. It is too late to change now, we shall have to continue with our right-hand screws, and the manufacturers will have to continue making and selling both right- and left-hand valves and hydrants.

To assure the opening of all gates shut down during a job, we require that the same man who shut down should open up, and that he keep an account of the number of gates he shuts to compare with the number he opens. In contract work the inspector must accompany the contractor's men from gate to gate and supervise their operation.

All our gates 16 in. and up have brick vaults and large sized covers. The idea is to have an interior space large enough to allow a man to repack the gate or change the screw, if necessary. Brick work is more flexible in a city where there are many underground structures. The vault can be shaped to fit the conditions found.

All deeply set gates have detachable extension stems to bring the operating nut to an easily accessible position below the surface. All our 36 in.

gates have indicators that show the position of the valve as it moves from open to close or vice versa. Square flax packing treated with tallow, or as within the last few years, with Py-Lubro, appears to give good results. In the old days we used tallow wicking. It packs easily and solidly, but eventually it frays, separates, and blows apart. Metallic packing is being used by several cities and towns. Time will tell if it is durable and efficient.

We use no gate less than four inches in size. If the pipe line is three inches in diameter, we increase at the gate and then reduce. We found that 2- and 3-in. gate stems, operated by the same wrenches that are used on 8-, 10- and 12-inch gates, broke easily. You cannot get a workman to adjust the output of his strength to the size of the gate. Your gate cannot be too rugged. It gets a great deal of abuse. In watching men operate gates, I have noticed an almost invariable tendency to open up hard. After the gate is well up and has indicated its extreme opening, they will make one or two powerful attempts to turn it more. One has to admonish them.

Gates leaking because of rust in the seats, are a nuisance, a cause of delay and an impediment to good work where one is making a connection or a repair. Our method of prevention is frequent operation, as a matter of inspection and maintenance, and running the gate all the way down and up several times before breaking out. If the gate continues to leak after breaking out, we shut behind it and patiently keep running it up and down until it becomes tight.

When snow and ice cover the ground it is important that, at least, our large gates should be quickly accessible. It is, therefore, our practice to spread coarse salt on the covers. We do the same on smaller gates on important connections. If the snow lasts for any length of time we continue with the work, especially in the outlying districts. During some winters salting has been quite an item in our maintenance work. A broken hydrant requiring the shut down of a main line with numerous side connections will give one a little worry if the controlling gates have to be excavated in deep snow and hard ice.

Where a system has several makes of gates, it is good policy to keep a stock of parts on hand. While Boston has gates of its own design, manufactured in its own shops, we have a number of gates that we purchased as part of tapping sleeves. We find a variation in these gates especially in the screws and expanding mechanism of the double discs. When the screw or any other part of the mechanism breaks or becomes inoperative, we are enabled to make quick repairs by having an assortment of spare parts on hand.

A considerable item of work during recent years has been the regulating of valve boxes to new grade where streets are accepted and permanently paved, or where pavements are relaid. In almost every case there is a slight change in grade with consequent necessity for raising or lowering the gate boxes. It is sometimes possible to raise or lower an iron screw box, but many times it is so rusted that it cannot be turned. In such a situation we use a

flanged frame of our own design and make, which is placed over the top of the box and set to grade. Brick gate chambers require a little masonry work for adjustment to grade, and boxes made of concrete slabs of which we have many, require the flanged frame mentioned, in raising or lowering. If the box is to be lowered we break off the top ends of the slabs and set the flanged frame over them. A little masonry work is necessary to do a good job in setting this type of frame.

A recent trouble is that of noisy covers. The automobile tire of today has a wide tread. It is not filled hard with air as in the past. The very latest make, I understand, carries only sixteen pounds of air. Consequently, when they pass over our covers, a strong suction is set up and the cover is lifted in its frame and then falls back again with a sharp metallic noise that is quite annoying to light sleepers during the night. It may be that because our covers are rectangular in shape with sharp corners that they do not fit into the frames as tightly as round covers, but I have listened to round sewer and electric-light manhole covers jumping in their frames. We have tried many ways to keep the covers in their seats; we have changed them; we have ground down high spots and burrs, we have packed the space between the edge of the cover and frame with yarn and cement, clay, elastic cement, rubber, wood, etc. but all without permanent success. We have had the best results with the yarn and cement. In applying any remedy we have to keep in mind the necessity for a quick removal of the cover. If it were not for that, there would be several ways in which the cover could be made fast.

The marking of gate locations is not so simple a matter in the outlying districts of the city. Mains are laid long before the streets are accepted and paved. There are no street sign posts and few, if any, buildings. Under such conditions it is necessary to place marks on trees, stone walls, realtor's sign boards, or fences. If no suitable place is available, we set a piece of 4- or 6-in pipe in the ground and use it as a marking post. Wooden stakes are not satisfactory.

Some troubles occur in connection with gate valves in service that fortunately are rare. I have in mind a loose body or valve ring. Once in a great while the iron into which these rings are expanded corrodes; the ring becomes loose, and during the operation of the gate, is caught by the descending valve or retards the upward movement of the valve. A major operation is called for in such a case. The entire gate is cut out and replaced.

On rare occasions the packing between bonnet and body flanges, blows. It can be replaced in service. It is well to cover iron bolt heads and nuts with a protective paint, preferably red lead, before the gate goes into the ground. If the soil is corrosive the heads and nuts will corrode and sometime in the future there will be trouble. Whenever we salvage a gate and put it through the shop, it is given a good coating of red lead paint, both inside and outside.

In the long run it pays to build or buy a gate of simple design, good workmanship, strongly built, without stint of good bronze where such metal is called for, and prime quality materials in other parts,— a gate with a generous factor of safety. A well-made gate seldom wears out from operation. Like the human machine it needs exercise. If given proper attention, its active life should exceed that of the pipe line in which it is set.

JOINT–LEAKAGE TESTS ON 16–INCH BALL AND SOCKET PIPE AT HAVERHILL, MASS.

BY HERBERT C. CROWELL.*

[*Read February 8, 1933.*]

The leakage tests reported in this paper were conducted on a 16-in. ball and socket pipe crossing the Merrimack river to connect the Haverhill high-service system with the Bradford water system. The crossing is located a short distance west of the County bridge and connects the 12-in. high-service pipe on Merrimack Street on the Haverhill side of the river with the 12-in. pipe in Main Street, Bradford. The line is 1 203 ft. in length and, with the exception of about 10 ft. of 12-in. pipe used for making the connection on Merrimack Street is 16 in. in diameter. The 10 ft. of 12-in. and 545 ft. of 16-in. pipe are of the bell and spigot type. The remaining 648 ft. of pipe are of the ball and socket type.

The ball and socket pipe was laid in that portion of the line extending across the river; the bell and spigot pipe was used in connecting the crossing with existing pipes on both sides of the river.

In order to comply with the War Department and the Department of Public Works regulations, it was necessary to lay the pipe through a portion of the river so that the top of the pipe would be 12 ft. below mean low water. To meet these requirements, it was necessary to excavate a trench from three to five feet in depth. In preparing a profile for the proposed pipe line, soundings were taken and test wells driven at intervals of 50 ft., except from Station 5+50 to 6+75. Between these stations the distance was reduced to 25 ft.

Three 16-in. gates were installed on the pipe line, one on the Haverhill and two on the Bradford side of the river.

The crossing is located in the central portion of the two distribution systems, and will allow turning the water from the Haverhill system direct into the Bradford system. As the reservoir on Gale Hill in Haverhill is 24.38 ft. higher than Dead Hill Reservoir in Bradford, the static head in Bradford will be increased by that amount. In case of a large fire the additional head available through the short connection of 16-in. pipe will add materially to the efficiency of the system. The main benefit of the connection is, however, the fact that it connects the two systems and in case of emergency can be used to strengthen either system.

*Superintendent of Water Works, Haverhill, Mass.

The pipe line was completed on September 1, 1928. Leakage tests were started on September 6, 1928, the following leakage being recorded with a static pressure of 115 lb. per sq. in.

Date.	Leakage. Gallons Per Hour.	Gallons Per 24 Hours.	Date	Leakage. Gallons Per Hour	Gallons Per 24 Hours.
Sept. 6	106	2 544	Sept. 18	12.5	300.0
Sept. 7	70	1 680	Sept. 19	11.6	278.4
Sept. 8	54	1 296	Sept. 20	11.6	278.4
Sept. 10	40.8	980	Sept. 21	9.4	225.0
Sept. 11	40.6	975	Sept. 22	6.0	144.0
Sept. 12	37.5	900	Sept. 23	4.0	96.0
Sept. 13	34.4	825	Sept. 24	2.0	48.0
Sept. 14	24.0	576	Sept. 25	1.0	24.0
Sept. 15	23.4	561	Sept. 26	0.5	12.0
Sept. 17	18.8	451.0	Sept. 27	0.0	0.0

The meter recording the leakage volume remained on the line for six weeks after the test was completed, but no loss of water occurred during this time.

HISTORICAL NOTES ON A LINE OF 10-INCH CAST IRON PIPE LAID IN 1854, PENNICHUCK WATER WORKS, NASHUA, N. H.

BY D. C. CALDERWOOD.*

[*Read March 8, 1933.*]

Last fall, the city of Nashua decided to resurface a section of its main street, and to cover abandoned trolley tracks, which for some time had been an obstacle to the smooth flow of traffic. A portion of this area to be resurfaced lay over 16- and 10-in. parallel water lines, while at either end were mains of much larger capacities. The Directors of the Pennichuck Water Works, looking toward the future demands of the distribution system, decided to replace the 10-in. line, laid in 1854, with a line of 24-in. cast-iron pipe.

One year before Nashua was incorporated as a city, and nearly twenty years before the larger neighboring cities attempted any such stupendous undertaking, a group of far-sighted citizens in Nashua associated themselves and in 1852 formed a company, the purpose of which was to secure an adequate supply of water for the thriving community. In the year 1854, the year this 10-in. pipe was laid, the first water was pumped into the system.

A schedule of proposed pipe lines is shown in our records of November 25, 1853, and a few days later authority was granted to purchase the pipe, "based upon the proposition of the Williams, Bird & Co., or upon such terms as may be for the best interest of the Company." The foundries of the Williams, Bird & Co. were located in Nashua, N. H. and Chelmsford, Mass., and the pipe was cast in Nashua. At that time Nashua was one of the leading iron and steel centers of this country, one of its foundries covering over fifteen acres. Up to this time, most of the iron and steel used in the United States was imported from Europe, and history records that the foundries in Nashua form a very important link in the development of the "Iron Age" in this country. Moreover, it might be of interest to note that during the Civil War, the port stoppers of the *Monitor*, the armored ship of the Union forces that gave battle to the Confederate *Merrimac* in the first naval engagement between two iron-clad vessels in the history of the world, were cast in Nashua. A brass marker was erected a few years ago to mark the spot occupied by these once famous forges.

To illustrate the difficulties encountered by the water-works pioneers, I will again quote from the records. In December, 1853: "a committee was appointed to determine the length of the socket of the pipe"; again in the same year a committee was chosen, "to make investigations as to the best material for packing the pipe"; (this evidently means caulking). These two

*Engineer, Pennichuck Water Works, Nashua, N. H.

notes in themselves bring back to our minds the problems encountered by these pioneers and the fact that water-works engineering was not always the highly standardized science that it is today. Evidently each community had to do its own research and experimental work, and it was not until many years later, after the formation of organizations such as this, that any semblances of standard specifications and practices were decided upon. (The New England Water Works Association standard specifications for cast iron pipe were adopted in September, 1902, forty-eight years after this pipe was laid.)

The original bill for this pipe, as well as the first and every subsequent bill ever paid by the water works, is on file at our office.

The prices quoted by the foundry in 1854 were 3½c. a pound for fittings and as follows for pipe:

Pipe size inches	Cost per foot
6	$0.80
8	1.20
10	1.50
12	2.00
14	2.70

Comparing these prices with the present day Class "C" pipe, which is what we use, we would obtain a figure of approximately $45 per ton. With reference to prices, another figure taken from the records in March, 1854 may be of interest. The proposal of Joseph F. Andrews was accepted, "which was to do all the digging as per specifications for the sum of $1.75 a rod." The specifications are not given, so it is not known whether this included excavating, laying, and backfilling, or not. But imagine today trying to excavate a trench by hand labor for a little over 10½c. a foot.

The pipe had no protective coating and was cast in a horizontal position, rather than vertically as is the practice today. On every length it is possible to determine two longitudinal lines, one on either side of the pipe, where the upper and lower halves of the flask came together. The iron itself was comparatively fine-grained, and the castings were symmetrical in shape, there being no undue variation either in the thickness or roundness of the pipe. The standard laying length was 9 ft. with a 5-in. bell, as compared to the present-day standard 4-in. bell, making the overall length 9 ft. 5 in. The following are the average dimensions taken from two lengths of the pipe:

	Ft.—In.
Overall length	9– 5
Laying	9– 0
Inside diameter	10
Outside diameter	11
Thickness	½
Depth of bell	5
Inside diameter of bell	11½
Outside diameter of bell	12½

The pipe weighed 510 lb. or about 57 lb. per ft. As previously mentioned, the bell was a full inch deeper than today's standard, but it had only a very slight groove one inch from the end instead of the ¼-in. groove that we now use; also there was no bead on the spigot end. Another interesting difference is that the outside diameter of the pipe was only ½-in. smaller than the inside diameter of the bell, thus leaving only a ¼-in. space for the joint and caulking. This evidently was not enough, because the joints were not very uniform; sometimes the depth of the lead would vary from one inch to four inches in the same joint. As far as it is known, no trouble was ever experienced with these joints, but it stands to reason that a more uniform joint is to be desired. (See Fig. 1.)

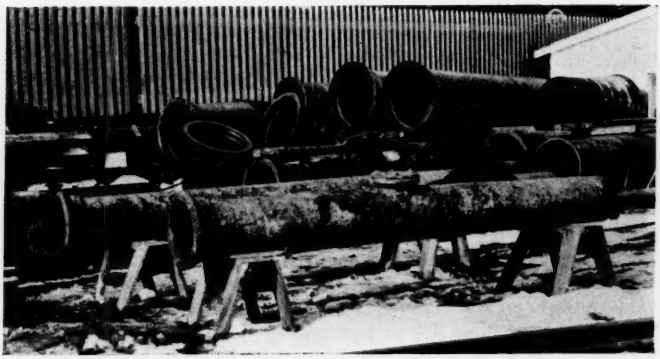

FIG. 1.— VIEW OF 10-IN. CAST-IRON PIPE LAID IN 1854 AT NASHUA, N. H.

The thickness and weight of this pipe conform somewhat to the present-day Class "A" pipe recommended for pressures not exceeding 43 lb. a sq. in., but this pipe had a pressure of at least 50 to 55 lb. per sq. in. on it at all times, and occasionally it has been raised to 75 or 80 lb. per sq. in. during a fire. No records can be found showing that this pipe ever failed, nor, with one exception, did we find any indications of repairs ever being made to this line. The exception was a longitudinal crack that had been repaired by bending a thin piece of metal, to the same shape as the outside of the pipe, placing what evidently was white lead between the pipe and this piece of metal and then strapping them securely together. This evidently is a satisfactory way of repairing such a leak, because it was placed there before any of the present-day employees worked for the company, and some of them have seen nearly thirty years of service.

All laterals were connected to this main by flanged joints, and flanges were used in connecting to the valve that was found in this line. These

flanges for the laterals were cast in the regular 9-ft. length, the flange for the branch being the only change from the regular pipe.

The 10-in. gate, which was of the single-disk rising-stem type, opening to the right, was made from two iron castings bolted together, grooved so that the disk could slide up and down. The disk and stem were securely fastened together and were operated by a brass nut that was held in place by castings fastened to the body of the valve. The disk consisted of a cast-iron circular plate with a bronze ring cast around it. The stem was of bronze and was cast with, and as a part of, the bronze ring around the disk. Other than this bronze ring and stem, the valve was made of cast iron, and in spite of

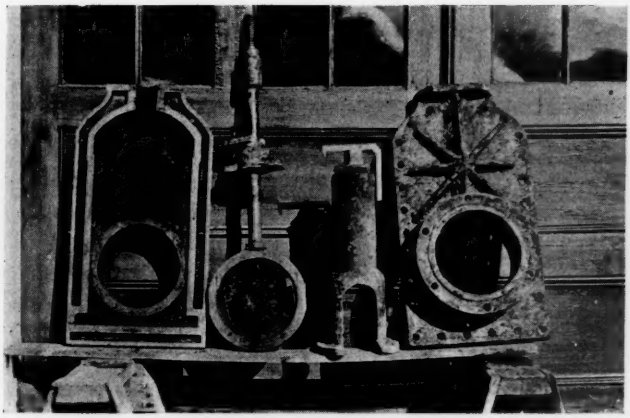

FIG. 2.— VIEW OF 10-IN. GATE VALVE TAKEN FROM 10-IN. PIPE LAID IN 1854 AT NASHUA, N. H.

the fact that it had not been used for over 25 years, the workmen were able to close it over one-half of the way, or, as far as the threaded section of the stem would permit. Years ago, about one foot of this threaded section had been broken off. This evidently was the reason why the valve was abandoned. (See Fig. 2.)

A 4-in. valve was found connected to one of the laterals, and while the cast-iron body was of the same general shape as that of the 10-in. valve, the operating mechanism was more like the valves used today. The disk was solid brass and was not fixed solidly to the stem as in the 10-in. gate. One remarkable point was the condition of the bolts and the ease with which the nuts were removed. After a slight tap of a hammer to loosen the rust, the nuts were easily turned off with a wrench. The bolts seemed as strong as ever, for they had only a very thin coating of rust, in spite of the fact that

they had been buried in the ground for 78 years. These valves were made by Gage, Warner, and Whitney, formerly local machinists and inventors, the prices ranging from $24.00 for a 4-in. valve to $75.00 for a 14-in. valve.

The carrying capacity of the pipe was almost as good as when it was laid. With only a very small amount of tuberculation or sediment in the pipe, the effective diameter was about the same as that of a 9¾-in. pipe. Had it not been for the anticipated progress and growth of the city, this pipe would still be rendering satisfactory service today.

FIELD TESTS OF LARGE METERS.

BY NELSON BOARDMAN.*

[*Read March 8, 1933.*]

The town of Danvers, having a population of 13 000 inhabitants, and a consumption of approximately 500 m.g. per year, considers it of utmost importance that all meters register as nearly as possible 100 per cent. of the water passed.

We have several institutions and industrial plants which use about 50 per cent. of the total water we pump; in fact, we have one institution that used 27 per cent. of our water last year, and another that used 13 per cent. At the present time the larger industrial plants are using about 15 per cent. of the water. There are twelve meters that register about 250 m.g. per year, and it can be readily seen that it is important to keep these meters in good running order.

We test all of our large meters twice a year, and if at any time we suspect that a meter is not registering correctly we give it another test.

We use rather an elaborate meter setting for our large meters, which consists of necessary fittings to by-pass the meter with test valve and tee. This allows us to clean and test the meter at our convenience. If for any reason it is necessary to take a meter out and send it back to the factory we can do so without interruption or inconvenience to the customer.

We have a field testing apparatus which includes a 2-in. disk meter by-passed with a ¾-in. disk meter, and is fed by a standard fire hose from the test tee of the meter to be tested. Both lines are controlled by quick-closing valves. On the discharge end, a multiple valve of orifices of different size is used, and this is placed higher than the meters so as to create a back pressure on the test meters. This testing device is capable of discharging about 200 g.p.m., which, in my opinion, is ample to test most any meter and affords us the opportunity of getting the same delivery year after year. This method of testing meters in the field has proved very satisfactory and has met with the approval of several meter manufacturers.

In my opinion most water works use meters of too large a capacity. There is a special recording and indicating device produced by one of the meter companies which affords the opportunity of finding out just what the maximum delivery is on any metered service. We ran tests on all our large meters with this device and the results were very interesting. We found out that many of our meters were too large and took steps to correct that fault. In some cases there were fire hydrants on these lines; therefore the size of the meter could not be reduced.

*Assistant Superintendent, Danvers Water Works.

On one of the institutions which has an 8-in. compound meter we found that at 3:00 P.M. they were using 40 g.p.m., and at 3:00 A.M. they were using 24 g.p.m. with a maximum demand of 74 g.p.m. It follows that if it were not for fire service we could use a meter of much smaller capacity.

At another institution where we have a 6-in. compound meter we found that at 3:00 P.M. they were using 23 g.p.m., and at 3:00 A.M., only 5 g.p.m., with a maximum use of only 29 g.p.m., but on account of hydrant service and being their only source of supply we could not reduce the size of the meter.

We had a 4-in. meter on one of the smaller industrial plants from which the average quarterly revenue was $41.00. This meter was tested and found to be slow. We replaced it with a 3-in. meter, and the first quarterly meter bill was $514.05.

In testing a large meter we allot the entire day, but generally this work can be completed in five or six hours. The meter testing device is operated by two men using a truck. The first thing we do is to run a test on the meter as we find it, through the multiple valve from a 2-in. stream to a ⅛-in. stream. We next take the meter apart and give it a thorough cleaning, making adjustments if necessary, and then run another complete set of tests. Sometimes we have to take the meter apart again and put in new parts or change the gears to obtain the desired accuracy. The test must be within four per cent. to be satisfactory and called final.

In the spring the method just described is applied, but in the fall we only test for accuracy. If found satisfactory we do not clean the meter.

All of our large meters are read and billed monthly. Comparative monthly readings are not sufficient to determine the accuracy of a meter, as we have found out from experience in the past. The only way to obtain satisfactory results is by flow tests.

I believe all meters should be owned by the water departments and serviced by them without cost to the customer, which in our particular case is covered by the much talked of service charge.

The most important things to take into consideration when metering a large water taker are the size and type of meter to be used. That is the reason why the water department, naturally being better versed on meters, should have control of them. If the consumer was to have control of the size and type of meter to be used, he would have every reason to select one which economically would benefit him most. He would purchase a meter not suited for the service required of it, from the water department's point of view.

I would like to say in closing that in my humble opinion most water departments, both municipal and private, spend too much time on small meters and not enough on their larger ones.

SOME ELECTRICAL KINKS IN WATER WORKS PRACTICE.

BY J. H. READ.*

[*Read March 8, 1933.*]

The primitive man who first rubbed a piece of amber across his furskin coat and in so doing discovered that it would pick up dry bits of wood, though only playing, was taking the first essential step in the conquest of that mightiest force of Nature called Electricity. That we know perhaps little more than this primitive man as to just what this force is, is not important. That we can master its rules of conduct that lead to the manifold daily uses so evident about us *is* important.

The electrical trade has made available many indispensible aids to modern practice in our field, but there still remain many simple ways in which we may help ourselves by the application of the bare rudiments of electrical knowledge.

A few years ago a representative of one of the larger recording gage and instrument manufacturers was visiting our office when our conversation was interrupted by the staccato ringing of a bell, a bell which to our office force meant that it was time to change the 24-hour recording pressure gage chart. But to our sales friend it evidently meant more than that for he got out pencil and notebook and made a sketch of the simple bell and battery circuit layout. Perhaps I should have called this visitor of ours a sales-engineer for not long thereafter this same scheme was presented in his company's catalogue, the only change being that the battery and bell circuit was housed inside the case.

Of late years, we have found many cases of ground wires being placed on the house side of meter settings, and quite frequently members of our meter-changing crew have received a shock when breaking the meter coupling. They are instructed to turn in a report on all such cases and change the meter only after all ground wires have been changed to the street side of the meter.

We started using jointing compound in place of lead a dozen years ago. A few years later we purchased an electrical thawing machine. You probably can anticipate the result or rather lack of result when we had occasion to thaw a service through a compound-poured line. We got advice from several superintendents and used a lead wedge for conductivity purposes between the bell and spigot after the joint was yarned. One of the superintendents told us he had been doing this for some time but had had no occasion, as yet, to thaw out a line so treated. We laid a stretch of 16-in. pipe with lead wedges and hauled the thawing machine onto the job although it was 100° F. in the shade at the time. Imagine our surprise at

*Assistant Superintendent, Manchester, N. H. Water Works.

being unable to build up a current. Our advisers, no doubt gave us credit for knowing enough to scrape off the coating of the bell and spigot at the point of application of the wedge. At any rate, that is all that was preventing an adequate thawing current.

We have a relatively small high-service distribution reservoir about three and one-half miles from its pumping station. A mechanical transmitter at the reservoir indicates and records the reservoir level at the station over a two-wire line. By using one of these lines and the ground for return, a relay was placed in this circuit to ring an alarm at the station when full reservoir was reached, thus affording a definite and independent check on the recording apparatus which is quite apt to get out of step due to various causes. We have also arranged a low-water emergency alarm in the superintendent's residence.

One evening last winter I received an urgent phone call from up state to inquire if I could locate beyond doubt a pipe line from a spring to a house and testify in court the next day as to this pipe line's proximity to a power pole carrying lightning arresters and a transformer. A power company was in the midst of a suit involving a death due to lightning coming in on their wires while the victim was drinking at the sink. Tracing this line with the pipe locater was simplicity itself compared to convincing a jury of twelve men tried and true that anyone could be sure where a pipe was without digging it up, frost or no frost.

The fact that salt in solution increases the electrical conductivity of water was made use of by Professor Allen and Mr. Taylor of Worcester to develop a method of water measurement called the salt-velocity method. We made use of this method at our low-service hydraulic station a few years ago in the development of characteristic curves for our water wheel. The method is simplicity itself and believed to be accurate. Salt was introduced near the upper end of the penstock and its passage across an electrode near the wheel recorded on a recording ammeter. The passage of salt between these two points was accurately timed and the volume of the penstock between the same points accurately determined. The discharge in cubic feet per second, of course, equalling the volume in cubic feet divided by the time in seconds. It seems to me that this method of water measurement has a great deal wider application than has as yet been accorded to it.

CONSTRUCTION OF THE COBBLE MOUNTAIN DIVERSION TUNNEL.

BY HARRY H. HATCH.*

[*Received March 23, 1933.*]

The Cobble Mountain Diversion Tunnel was almost the first structure to be built in connection with the water-supply and power-development project of the city of Springfield on the Westfield Little River.

The reasons for construction of this tunnel were;

(1) To divert the flow of the Little River during construction of Cobble Mountain Dam.

(2) To control the 1½ billion gallons of water stored in the reservoir below the draft line of the power tunnel at El. 830, the city holding this water in reserve for use in extreme droughts.

(3) To aid, if necessary, in discharging extreme floods. This is a remote possibility, as the maximum flood experienced during the last twenty-five years has been 5 800 sec. ft., whereas the spillway of Cobble Mountain Reservoir has a capacity of 20 000 sec. ft., without taking into consideration the discharge capacity of the power tunnel which can be used in relieving flood dangers.

Location. The entrance portal of the tunnel lies about 1 100 ft. upstream from the upper toe of Cobble Mountain Dam (Fig. 1). The tunnel runs eastward in a straight line and cuts off a bend in the river. Its lower portal is about 150 ft. below the downstream toe of the dam. While the entrance could have been placed nearer the dam, slightly shortening its length, there was good reason for keeping the entrance well out of the way. There is always a chance that a slide will occur during construction of a hydraulic fill dam, and a slide becomes especially dangerous when the dam has the unprecedented height of 263 ft. If the mouth of the tunnel is too near the dam, a major slip during construction may carry core material into the tunnel and render it useless. This happened some years ago during construction of the Calaveras dam.

Description of Tunnel and Shaft. The tunnel proper is horse-shoe shaped, 11 ft.-6 in. high and 11 ft.-6 in. wide with a cross-sectional area of 113.3 sq. ft., equivalent to a 12 ft. circular section. It is lined with concrete throughout its length. The entrance section, 25 ft. long, is flared and has a level grade. The portal opening, 360 sq. ft. in area, is 20 ft. wide with vertical side walls rising 10 ft. 2 in. to the springing line of a semi-circular arch of 10-ft. radius. The tunnel tapers to the normal section within the

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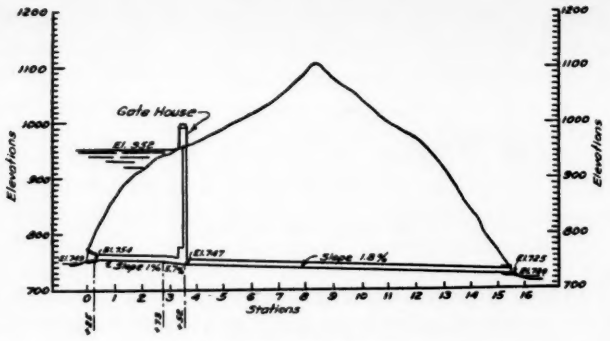

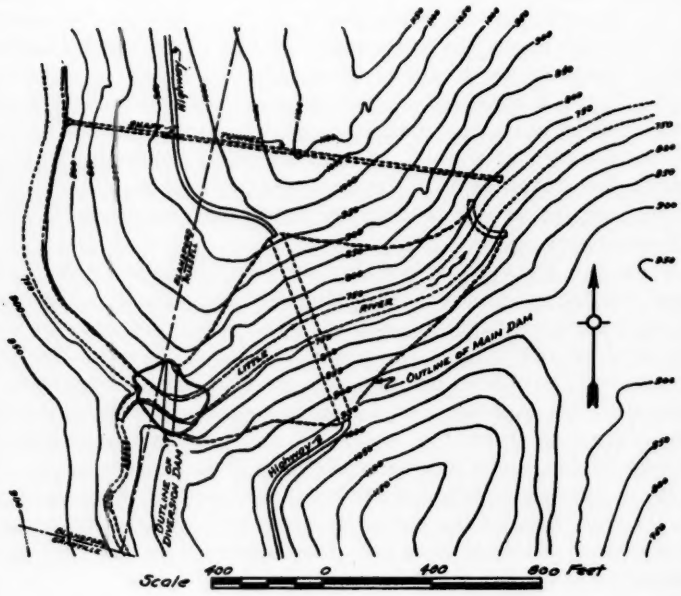

FIG. 1.— PLAN AND PROFILE OF LOCATION OF COBBLE MOUNTAIN DIVERSION TUNNEL.

flare distance. The elevation of the tunnel entrance and the flared section is 754 ft., and is followed by a 1 per cent. slope for 240 ft., a 5.7 per cent. slope for 79 ft. to the shaft and finally a 1.8 per cent. slope to the lower portal, ending at El. 725. The length of the tunnel is 1 550 ft.

At the bottom of the shaft there is an 18 ft. by 24 ft.-8 in. gate chamber about 22 ft. high. Above it lies a control chamber 14 ft.-8 in. by 18 ft., about 10 ft. high. Both of these chambers have flat arched roofs. The shaft is lined with concrete, has a net cross-section of 10 ft. by 10 ft., and is 219 ft. deep to the floor of the gate chamber. The opening is divided in half by an 8-in. concrete partition wall. The westerly half contains the concrete stairs which lead from the surface to the control chamber; the easterly half is left open to serve as an air vent and for other purposes such as the lowering of equipment. The gate chamber section is tapered to the normal section of the tunnel, in a distance of 8.5 ft.

About 300 ft. below the tunnel entrance, near the shaft, a 45-ft. length of tunnel was left unlined to receive a concrete plug after completion of Cobble Mountain Dam. This plug is pierced by two 42-in. pipes which lead to the gates at the bottom of the shaft. These gates may be opened for the reasons previously stated, and also for periodic test to see that they function properly.

The diversion tunnel, before being plugged, had a discharge capacity of 4 000 sec. ft. with the pool flow-line at El. 810.

The Diversion Dam. In connection with the tunnel, a rock-fill diversion dam was built which later became the upper toe of Cobble Mountain Dam. This dam is about 75 ft. high, 240 ft. long at the top and 70 ft. long at the bottom. It has a width of 40 ft. on top and about 300 ft. at the bottom. In order to utilize the tunnel muck without obstructing the river, a crib work was built near the northerly bank of the gorge within the outline of the dam, leaving a sufficient opening near the southerly bank of the river to take care of the flow during construction of the dam. The cribbing was filled with about 15 000 cu. yd. of tunnel muck. The diversion dam has a rolled earth apron on the upstream side varying in thickness from 10 ft. at the top to about 25 ft. at the bottom and serving to control the leakage of water through the rock fill during construction of the main dam. The diversion dam has a total volume of 55 000 cu. yd. About 34 000 cu. yd. of rock were borrowed from nearby quarries to complete the structure. The top of the diversion dam is at El. 810 or 56 ft. above the invert of the tunnel entrance.

Driving the Tunnel.

Tunnel Excavation. Most of the tunnel was driven down grade from the entrance portal heading, except near the outlet where about 100 ft. were bored through from the lower portal. Throughout its length the tunnel penetrated solid rock, the characteristic mica schist of this region. During excavation of the first 400 ft. of the tunnel, the contractor had several superintendents who tried different schemes of drilling and shooting the heading. These schemes consisted in the variation of heading and bench methods (Table 1).

TABLE 1.—DRILLING AND SHOOTING SCHEDULE FOR VARIOUS TUNNEL — HEADING AND BENCH METHODS.

Scheme No.	1	2	3	4	
From Station	0+10	1+10	2+03	2+39	3+66
To Station	1+10	2+03	2+39	3+30	3+92
Drilling:					
Number of holes drilled					
Total	47	40	44	53	
Cut	7	7	10	13	
Reliever	6	6	6	6	
Breakdown	4	4	4	6	
Square-up	12	12	9	8	
Down holes (Bench)	18	..	..	20	
Lifters (Bench)	..	11	15	..	
Length of holes in feet					
Heading					
Cut holes	8	8	8	8	
Others	7	7	8	8	
Bench					
Down holes	6	..	..	4	
Lifters	..	12	12	..	
Advance in feet per round					
Heading	5	5	5	7	
Bench	6	10	10	7	
Linear feet drilling per cu. yd. excavation	10.6	9.7	11.3	8.7	
Shooting:					
Heading					
Volley 1 Direct	1– 7	1– 7	1–10	1–13	
Volley 2 1st delay	8–13	8–13	11–16	14–19	
Volley 3 2nd delay	14–17	14–17	17–20	20–25	
Volley 4 3rd delay	18–29	18–29	21–29	26–33	
Bench					
Volley 1 Direct	1– 6	30–35	30–34	16–20	
Volley 2 1st delay	7–12	36–40	35–39	11–15	
Volley 3 2nd delay	13–18	....	40–44	6–10	
Volley 4 3rd delay	....	....	....	1–5	
Pounds explosives per cu. yd. excavation	7.2	6.1	7.7	5.2	

Ultimately the full face heading method (Fig. 2) was adopted with a total of 43 holes. The cut holes varied from 13 to 10 ft., and the other holes from 12 to 8 ft., with resulting draw of from 10.5 to 7.5 ft. per round. From Station 8+10 on, for the remaining 730 ft. of the tunnel, the cut holes were 10 ft. long and the others 8 ft., averaging a draw of about 7.5 ft. per round.

The shooting schedule was as follows:

Volley No.	Hole No.	
1	1 to 12,	consisting of 10 cut and 2 buster holes located in the middle and near the floor of the tunnel. These were first shot independently.
2	13 to 20;	relievers outside the cut holes, were shot direct.
3	21 to 33,	consisting of 8 square-up holes on the sides of the tunnel to the springing line and 5 relievers or break downs in the arch, were shot with the first delay.
4	34 to 43,	square-up holes in the crown, were shot with the second delay.

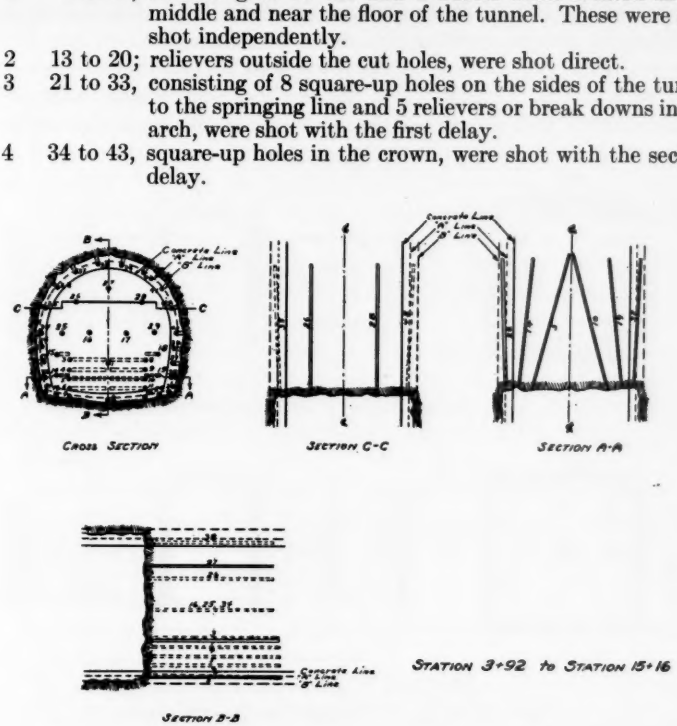

FIG. 2.— FULL FACE HEADING METHOD OF DRIVING TUNNEL.

Unexploded holes in No. 1 Volley were reloaded and shot with Volley No. 2. For the full face heading methods, the average length of drilling was 8.7 ft. and the consumption of dynamite was 5.2 lb. per cu. yd. of solid rock excavation. The average use of explosives for the entire tunnel was 6 lb. per cu. yd.

The reasons for adopting the full face heading method may be summarized as follows: The heading and bench method, as a rule, required more explosives and drilling per cubic yard of excavation than the full face heading method; the shot in the heading often piled up the muck on the bench and required hand shoveling before the drilling crew could start, thus causing delay in the progress of work. Usually headings and benches were shot separately; this doubled the time lost for removing the tools, etc. before shooting and for waiting after the shot to free the tunnel of fumes.

In general 60 per cent. Atlas gelatin dynamite was used in excavating the tunnel, with direct and several delay exploders. For a time the holes were tamped with tamping-bags filled with sand; later additional sticks of dynamite were used for this purpose.

A 1 000-cu. ft. Laidlaw stationary electric air compressor located at the surface near the top of the shaft, supplied the compressed air necessary for the drills, pumps, mucking machine, concrete gun, etc. A type No. 50 Ingersoll-Rand pneumatic drill sharpener with oil furnace was also used.

For a couple of hundred feet the headings and benches were drilled with ordinary "jackhamers." Beyond that the drilling was done with six

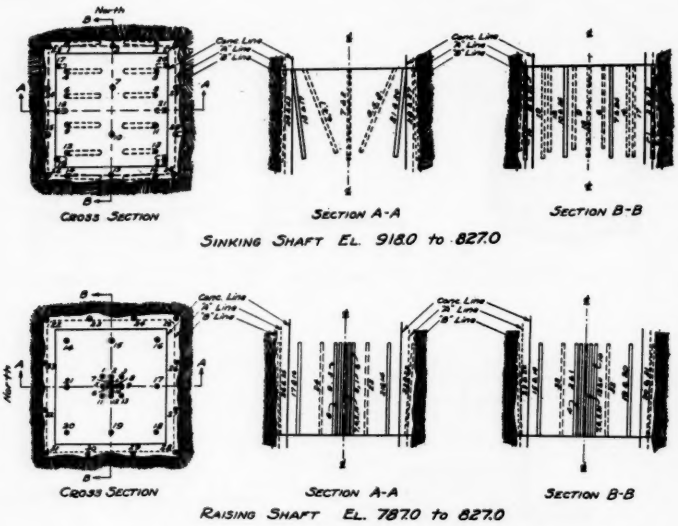

FIG. 3.— METHOD OF SINKING AND RAISING SHAFT.

Leyner drills mounted on the cross arms of two Ingersoll-Rand 12-ft. steel columns. A working platform was erected just above the springing line. Four Leyner drills were used for the holes below the platform, and two for the remaining holes. The drilling crew consisted of one foreman, six drillers and six dummies. A small Hoar mucking machine was used for loading 1-cu. yd. steel side-dump cars hauled by small Plymouth gasoline locomotives over a narrow-gage track.

The work schedule started with two 10-hour shifts per day when drilling and mucking were done together, as far as it was possible. For a time "call shifts" were employed, there being no definite starting time for each shift, the crew being called as soon as its particular work was ready. A crew thus worked one, two or more shifts per 24 hours. Later on, four 6-hour shifts per day were adopted; two for drilling and two for mucking. The drillers, even though working only six hours, were paid for ten hours at a rate of $1.00 an hour. Each crew was given so much to do on condition that it would not last more than six hours. When, however, a crew began to finish the given work in less than six hours, the required work per shift was

increased. The scheme worked well for a short time, but the additional requirement and also the drillers' efforts to get the required work done in the shortest possible time resulted in careless drilling alignment, especially of the square-up holes. This caused much overbreakage. When the bonus system was abandoned, two 10-hour shifts per day were adopted again. The general progress from the shaft onward for about 1 000 ft. averaged 12.2 ft. per day, the maximum being 22 ft. of tunnel excavation for a 24-hour period.

The shaft was completed during the early stages of the work, and acted as a vent for the tunnel. There was no need for artificial ventilation. After the headings were shot, it took about twenty minutes for the fumes to escape through the shaft. The men were not allowed to return to the heading until half an hour after the shooting.

When a tunnel is driven down grade, the question of drainage becomes important. The tunnel was comparatively dry; yet occasionally wet seams and a small amount of running water were encountered. This water added to that used in the Leyner drills required the constant use of an air-operated piston pump.

Shaft Excavation. Solid ledge was struck at El. 957, or 9 ft. below the top of the shaft at the surface. From this point the shaft was sunk to El. 827, a distance of 130 ft. From elevation 770 to 827, a distance of 57 ft., the shaft was raised, the excavation being embodied in that of the gate chamber. The two headings met perfectly.

Jackhamers were used for sinking the shaft and stopers for raising it. (Fig. 3.)

The drilling and shooting methods are given in Table 2, Nos. 1 to 4 inclusive for sinking, and Nos. 5 and 6 for raising the shaft:

TABLE 2.—DRILLING AND SHOOTING SCHEDULE FOR SHAFT.

	SINKING.				RAISING.	
Number of Holes.	No. 1 Elev. 957–935.	No. 2 Elev. 935–924.	No. 3 Elev. 924–918.	No. 4 Elev. 918–827.	No. 5 Elev. 770–787.	No. 6 Elev. 787–827.
Total	36	34	40	30	28	33
Cut	12	12	18	12	12	13
Buster	0	2	2	4	0	0
Relievers	8	6	6	6	4	8
Square-ups	16	14	14	8	12	12
Depth of holes — ft.	4	6	8	8 & 10	9	8 & 12
Depth of draw — ft.	2.5	3	6	6 & 8	7	7 & 11
Dynamite, per cent.	40	40	40	60	40	40 & 60
Pounds dynamite per cu. yd. rock	4.6	3.6	3.2	3.2	2.9	5.0

The general average of explosives used for the entire shaft was 4 lb. per cu. yd. of rock. Usually the shots were made in three volleys. The first volley included cut and buster holes, which are sometimes called sump holes in shaft excavation. The second volley covered the relievers and the third the square-ups. In some cases the sump holes were shot first and mucked out; then the other holes were loaded and shot. In sinking the shaft, the muck was loaded by hand into a yard bucket which was raised to the surface with a derrick, the material being wasted. The muck from the shaft-raising was handled in the same manner as ordinary tunnel muck.

Lining and Grouting Tunnel.

Concrete Lining. Concrete aggregate included:

1 barrel of Portland cement weighing 376 lb. net.
9 cu. ft. of sand measured loosely.
16 cu. ft. of coarse aggregate, measured loosely.

The cement was required to conform to the specifications of American Society for Testing Materials, except that the SO_3 content was not to be more than 1.5 per cent. The sand was required to be clean, free from organic matter and of hard durable stock, graded from fine to coarse, and passing when dry a screen having ¼-in. diameter holes and not more than 30 per cent. by weight passing a sieve having 50 meshes per lineal inch. The coarse aggregate was required to be clean, hard, durable, and of such size as to pass a 2½-in. ring, all particles smaller than ¼ in. being screened out. After the charge was completed a minimum mixing time of three minutes was required. An average of 6½ gal. of water per bag of cement was used.

The mixing plant was located at the top of the shaft. The concrete was mixed in a four-bag batch Ransome mixer operated by steam engine. Concrete was discharged into a chute in the form of an 8-in. diameter pipe about 200 ft. long, leading to a wooden hopper of about 2-cu. yd. capacity located near the bottom of the shaft. From the hopper the concrete was loaded into 1-cu. yd. steel side-dump cars. Concrete trains consisted of two or three cars which were hauled by a small Plymouth gasoline locomotive over the narrow-gage track to the place of pouring.

The invert was poured first, together with about 9 in. of the side walls. The wooden forms for these side-wall sections were suspended from the falsework along the plane of the springing line. These forms were held in place by wire tied to expansion bolts anchored in the rock on the sides. The floor of the tunnel was thoroughly cleaned to solid ledge before pouring concrete. No back-filling with muck, even in large overbreaks, was allowed. The concreting of the invert started from the east portal towards the shaft. A maximum of 158 ft. (80.0 cu. yd.) of lining and a general average of 47.0 ft. (41 cu. yd.) a day were obtained. A minimum time of 24 hours was required to elapse before track was laid on fresh concrete invert.

The side walls and arch were poured together by pneumatic process in 60-ft. sections. Collapsible Blaw-Knox steel forms and a Ransome concrete gun were used. The concrete cars were hauled over the incline and the elevated track, and dumped into a steel side platform right over the opening of the concrete gun. The concrete was shoveled by hand into the gun and shot back of the forms with about 100 lb. per sq. in. pressure through 6-in. pipe. Reinforced rubber pipes were used in the bends of the concreting pipe. They were good for only about 1 000 cu. yd. of concrete.

As a rule, forms were not stripped until after the concrete was 24 hours old. The general average progress for the side walls and the arch was 47.0 ft. (70.0 cu. yd.) per day.

After the lining was started, it was discovered that it would have been better to pour the side walls first to about the springing line and then shoot the arch. The concreting pipe was located right at the crown, and the concrete had to fall about 12 ft. along the arch and side walls. As the concrete was discharged from the end of the pipe, the large aggregates travelled faster and invariably formed honeycombs near the bottom of the side walls at the beginning of the new section. At the back of the forms, the concrete took a diagonal slope, from the crown to the bottom of the side wall. This slope, as a rule, was at an angle of about 30 degrees with the horizontal until the side walls began filling up to higher elevations. Along these slopes, there was a distinct segregation of coarse aggregates. At the beginning of each batch-discharge from the concrete gun, the larger aggregates would travel faster and deposit at the bottom of the diagonal slopes.

For a few sections of the tunnel, 3 per cent. celite was added to the cement, but with no improvement in the placing or in the quality of the concrete. The cost of celite per cubic yard of concrete was 38 cents. A richer mix of concrete, 1:2:3½ instead of 1:2¼:4, was tried. The cost of the extra cement per cubic yard of concrete was equivalent to that of the celite used. There was no improvement in the segregation of the aggregates, but the concrete in this case was denser than that made with celite. However, the original mix was continued with better results after strict inspection was instituted. It may be proper to say here that between a good and a poor inspector on a concrete job, the quality of concrete may vary from the best to the poorest. With due attention to placing, tamping, and spading, it was possible to obtain a good concrete with the original mix of about 1:2:4.

To do away with the high dropping of the concrete from the crown to the side walls, as a trial, the concreting pipes were brought through the panels of the steel form just above the springing line and discharged the concrete into the side walls first. The concrete was well spaded, and in addition the forms were tapped by pneumatic hammers from outside. Later the discharge pipe was brought to the crown, and the arch was poured.

In the first few sections, especially in the crown, the concrete did not

have a fine finish. It was peeled off. The reason for this was that the discharge from the concreting pipe hit the steel forms and scraped off the oil; when the forms were stripped part of the concrete lining came off with the forms wherever the oil had been scraped off. To remedy this situation, the direction of the discharge pipe was changed so as not to hit the inside face of the forms.

The contractor put in wooden bulkheads at the ends of the steel form sections for construction joints. The irregular breaks in the rock made it rather hard to have a tight form, and the work required almost the entire time of two carpenters.

The shaft was lined with concrete from the bottom up. The lining, partition wall, arched stairway and landings were poured in 10-ft. lifts. In

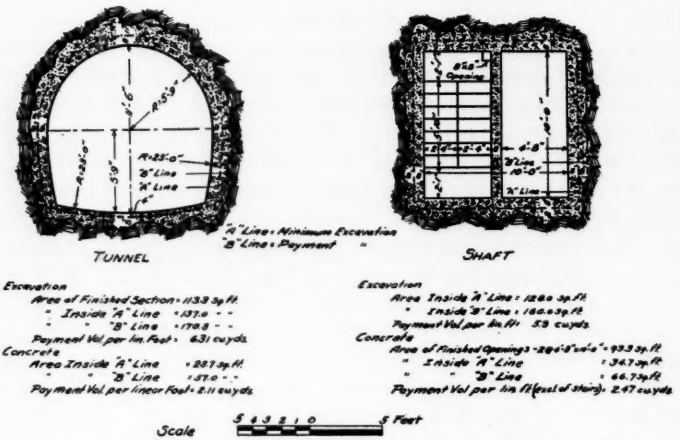
TUNNEL
SHAFT
"A" Line = Minimum Excavation
"B" Line = Payment
Excavation
Area of Finished Section = 113.3 sq. ft.
" Inside "A" Line = 137.0 " "
" " "B" Line = 170.8 " "
Payment Vol. per lin. Foot = 6.31 cu. yds.
Concrete
Area Inside "A" Line = 28.7 sq. ft.
" " "B" Line = 57.0 " "
Payment Vol. per linear Foot = 2.11 cu. yds.
Excavation
Area Inside "A" Line = 1280 sq. ft.
" Inside "B" Line = 1664 sq. ft.
Payment Vol. per lin. ft. = 59 cu. yds.
Concrete
Area of Finished Openings = 29-6'-8" x 6'-0" = 93.3 sq. ft.
" Inside "A" Line = 34.7 sq. ft.
" " "B" Line = 66.7 sq. ft.
Payment Vol. per lin. ft. (vol. of shaft) = 2.47 cu. yds.
Scale
5 4 3 2 1 0 5 Feet

FIG. 4.— CROSS-SECTION OF TUNNEL AND SHAFT SHOWING PAYMENT LINES AND QUANTITIES.

this case the concrete was placed directly into the forms from the 8-in. chute.

For construction joints and for repairs 1:2 mortar was used. Briquettes were made daily in the field, and a complete record of tests was kept. The following figures show the average tensile strength of all the field briquettes:

Age	Strength Lb. per sq. in.
24 hours	23
2 days	66
7 days	232
28 days	373

Payment Lines. The required minimum excavation (Fig. 4) as given by the "A" line, was 8 in. outside the arch and the sides, and 4 in. below the invert of the finished concrete section of the tunnel. However, payment was

made to the "B" line, which lies outside the "A" line by amounts varying from 11 in. at the crown to 8 in. at the springing line, and continuing at a uniform distance of 8 in. on the sides and the bottom of the tunnel. The difference between the "A" and "B" lines was 1.24 cu. yd. per running foot. Whatever the section of the tunnel excavation outside the "A" line, the contractor was paid only to the "B" line both for rock excavation and concreting.

In excavation and concrete lining of the shaft a uniform allowance of 8 in. was made outside the "A" line. This allowance amounted to about 1.2 cu. yd. per running foot of shaft.

Both in the tunnel and the shaft, the requirements of the specifications would have been fulfilled if excavation in rock had been made only to the "A" line and concrete had been carried from that line to the finished section. The allowances previously mentioned were made both for excavation and concrete to allow for overbreakage in the rock and for the concrete required to fill up the holes.

A tunnel pantograph was devised and constructed on the job. It was used to measure accurately a sufficient number of sections in the tunnel to permit finding the total excess excavation and concrete yardage outside the "B" line. Similar data were obtained for the shaft by offsets from the concrete forms.

TABLE 3.— COMPARISON OF ACTUAL AND PAID QUANTITIES OF EXCAVATION AND LINING.

	Per Linear Foot.	
Excavation.	Tunnel.	Shaft.
Minimum ("A" Line) — cu. yd.	5.1	4.7
Payment ("B" Line) — cu. yd.	6.3	5.9
Actual — cu. yd.	6.9	6.4
Excess outside "B" Line — cu. yd.	0.6	0.5
Per cent. Excess	10	9
Average overbreakage, outside "A" Line — %	13	11
Allowed overbreakage, outside "A" Line — %	9	8
Concrete Lining		
Minimum ("A" Line) — cu. yd.	0.9	1.3
Payment ("B" Line) — cu. yd.	2.1	2.5
Actual — cu. yd.	2.7	3.0
Excess outside "B" Line — cu. yd.	0.6	0.5
Per cent. Excess	30	20

Grouting. Before the lining was started, a careful survey was made of water conditions in the tunnel. All seams and mud pockets were carefully noted, with a view to filling them with grout later on. During lining operations drain pipes were placed through the forms to take care of the running water. These drains were later grouted.

An open space always remains between the concrete and the rock surface of a tunnel, especially at the crown. Before the concrete sets, it settles

and shrinks. This may be the cause of the space left back of the lining, but often some of this opening is due also to the fact that all the space inside the forms is not filled with concrete. As a rule the depth of the opening is greatest at the crown and disappears gradually about at the haunches of the arch. There are hardly any voids back of the concrete below the springing line.

The grouting schedule included three types: (Fig. 5).

Type I, to fill the voids at the crown of the arch.
Type II, to fill the voids back of the concrete at the haunches.
Type III, to seal the cracks and seams in the rock.

Particular attention was paid to grouting from Station 2+00 to Station 7+00. When the reservoir is full, the lining from the West Portal

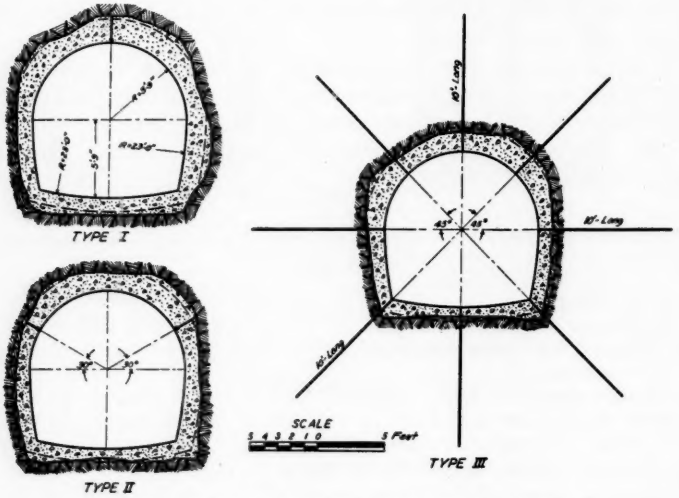

FIG. 5.— TYPES OF GROUTING EMPLOYED IN TUNNEL.

to Station 3+32 will be subject to a pressure head of about 200 ft. It was desirable to seal all the rock seams up to Station 7+00, which is well past the center-line of the dam, in order to stop possible leaks between the reservoir and the tunnel.

Operations for Type I grouting consisted of grouting the roof of the tunnel through one grout hole at various intervals. These grout holes were 20 ft. on centers from Station 0+20 to Station 1+80, and from Station 7+05 to Station 15+25; they were 10 ft. on centers from Station 2+00 to Station 6+85.

During the pouring of the first few sections of lining, 1-in. pipes with couplings at their lower end were inserted in the crown of the steel forms.

The pipes extended clear to the rock surface and were held rigidly so as not to be displaced during concreting. The pipes were filled with sawdust held in place at the bottom by waste, in order to prevent their being filled with mortar during the lining operations. It was soon discovered, however, that it was impossible to hold the grout pipes in place. Not only were they in the way of the concreting pipes during the pouring of the arch, but also they could not withstand the high velocity of the discharge from the concreting pipes. Hence it was decided to drill a hole in the concrete lining rather than to provide grouting pipes while placing concrete back of the forms.

Type I grouting was started from the East Portal; every fourth hole, 80 ft. center to center, was grouted first; then every half-way hole; finally the one-quarter holes, bringing the distances down 20 ft. on centers. Intermediate holes were then put in bringing the centers 10 ft. apart between stations 4+05 and 6+85. The first grouting was allowed to set before the next series was started.

Type II grouting consisted of grouting a hole on each haunch of the arch located radially 30 degrees above the springing line. This type was used at 10-ft. intervals between Stations 2+05 and Station 4+05, starting with the lowest elevation in the section and gradually working to the end. After the grout was set, Type I grouting was done within this section.

Type III grouting consisted of eight 10-ft. holes drilled through the concrete lining. These holes were located as follows:

Two holes, horizontal, one on each side at the springing line.
One hole, vertically up through the crown.
One hole, vertically down through the invert.
Two holes, one at the intersection of each side wall with the invert, 45 degrees below the springing line.
Two holes, through the haunches of the arch 45 degrees above the springing line.

Type III grouting was used between stations 1+95 and 4+00. Grout holes were put in wherever there were cracks and seams in the rock back of the concrete. The holes were drilled in such a way so as to catch the seams. There were 10 such groups of grouting holes. After these had been filled and the grout had set, similar groups were put in at intervals so as to bring them to 10 ft. centers within this section. Following this, additional intermediate groups were provided and grouted wherever it was deemed necessary.

In the gate chamber, grouting holes were used, wherever necessary, to fill up the cracks and seams in the rock.

Grout holes were 2 in. in diameter. The pipes were 1½ in. in diameter and about 12 in. long. About a 6-in. length of the pipe was inserted in the hole and calked tightly with oakum. The remaining 6-in. length had an outside thread at its lower end for connection to a coupling with an attached valve. The valve was connected to the end of the grout hose of the

grouting machine. After completing the grouting of each hole, the valve and coupling and the grout pipe were pulled out and cleaned thoroughly and used over again. The hole in the concrete was filled with mortar and pointed up.

The grouting machine consisted of a small cylindrical steel tank equipped with a flap valve in its cover, through which the grouting materials were introduced. The charge consisted of one bag of cement and about 8 gal. of water for neat grouting. One or two cubic feet of sand were added whenever sand was used in the mix. The hopper bottom of the tank opened into a 1½-in. pipe which served as an air inlet connection for mixing the grout for one or two minutes. This connection was used also to discharge the grout after closing the flap valve and opening the main air line. The discharge pipe was attached to the grouting hose leading to the grout pipe. The grout was shot under pressures up to 100 lb. per sq. in.

Every hole received neat grout at the start unless a big opening was known to exist. When the grout hole was observed to require a considerable quantity of grout, sand was added to the mix. However, no mix leaner than one part of cement to two parts of sand was used. The average mix employed in grouting the tunnel was about 1 to 1.

During the grouting of the first holes, the adjacent ones acted as vent pipes; but while the final holes were being filled there was no means of escape for the air. To vent the last holes, the pipes instead of being calked with oakum were held in place by small wooden wedges which left enough space between the pipes and concrete to act as an air vent. The wedges were removed with the grout pipe, and the holes pointed up as usual.

The amount of grout taken at various points in the tunnel was very irregular. This was due in part to the isolated occurrence of large seams and pockets, and in part to overbreakage during tunneling. In a mica-schist rock tunnel, the projection of sharp rocks interferes with the continuous flow of grout over any appreciable distance, especially when the grout contains sand. The fact that two holes, say, 50 ft. apart refused grout does not mean that all the space between the concrete lining and the rock surface of the tunnel between these holes is entirely filled with grout. In order to completely grout a tunnel, it is necessary to know accurately the required volume of the concrete lining in short sections of the tunnel and how much concrete has actually been placed. If the amount of grout used is recorded and due consideration is given to seams in the rock it becomes possible to know whether or not grouting has been carried to completion.

Grouting was done as extra work on a cost plus 10 per cent. basis. The net cost of Type I and Type II grouting was $5.03 per running foot, and of Type III $3.75. In Table 4 are given various data on quantities and net costs of grouting.

TABLE 4.—EXTENT AND COST OF GROUTING.

	Quantity.	Cost — $
Number of grout holes	417	
Net cost		9 430.62
Net cost per hole		22.62
Cement used — bags	3 818	
Sand used — cu. ft.	4 266	
Cost per bag of cement		0.73
Cost per cu. ft. of sand		0.082
Cost of handling cement per bag		0.16
Cost of handling sand per cu. ft.		0.16
Average labor charge per hour		0.60
Compressor rental charge per hour		1.925
Grout machine rental charge per hour		0.125
"Jackhamer" and steel rental charge per hour		0.10
Compressor hours	474.5	
Lineal feet drilled	2 361	
Lineal feet drilled per hour	5.3	
Cost of 1½ in. pipe per foot		0.135
Labor hours	6 225	
Number of grouting days	62	
Material per hole — cu. ft.	18.0	
Ratio of grout to concrete in tunnel lining — %	6.7	
Amount of grout — cu. yd.	280	
Net cost of grout per cu. yd.		33.68
Bags of cement per cu. ft. grout	0.505	

TUNNEL APPURTENANCES.

Control Gates. In the summer of 1931 the dam had reached an elevation of about 900 ft., and thus provided sufficient reservoir capacity to store all ordinary floods. While the flow of the Little River was at a minimum, therefore, the diversion tunnel was plugged.

There were installed within the concrete-plug section two 42-in. steel welded pipes with field-riveted transverse joints. Before concreting, the pipes were held to true alignment and grade by false work. By means of a special section each pipe was connected to a 39⅜-in. Escher-Wyss rotary gate valve, which in turn was connected by means of another special section to a 42-in. by 30-in. Larner-Johnson discharge regulator, all within the gate chamber (Fig. 6).

Temporary bulkheads were installed while the concrete in the flared sections at the entrance to the pipes was being poured for one pipe at a time, the flow being carried through the other pipe. The length of the flared section from the end of the pipes to the normal section of the tunnel was 8.5 ft.

The concrete plugging was done in about 10-ft. sections beginning at the upstream side. From the crown of the section of previous pour, 1½-in. grout pipes were extended through the next section. These

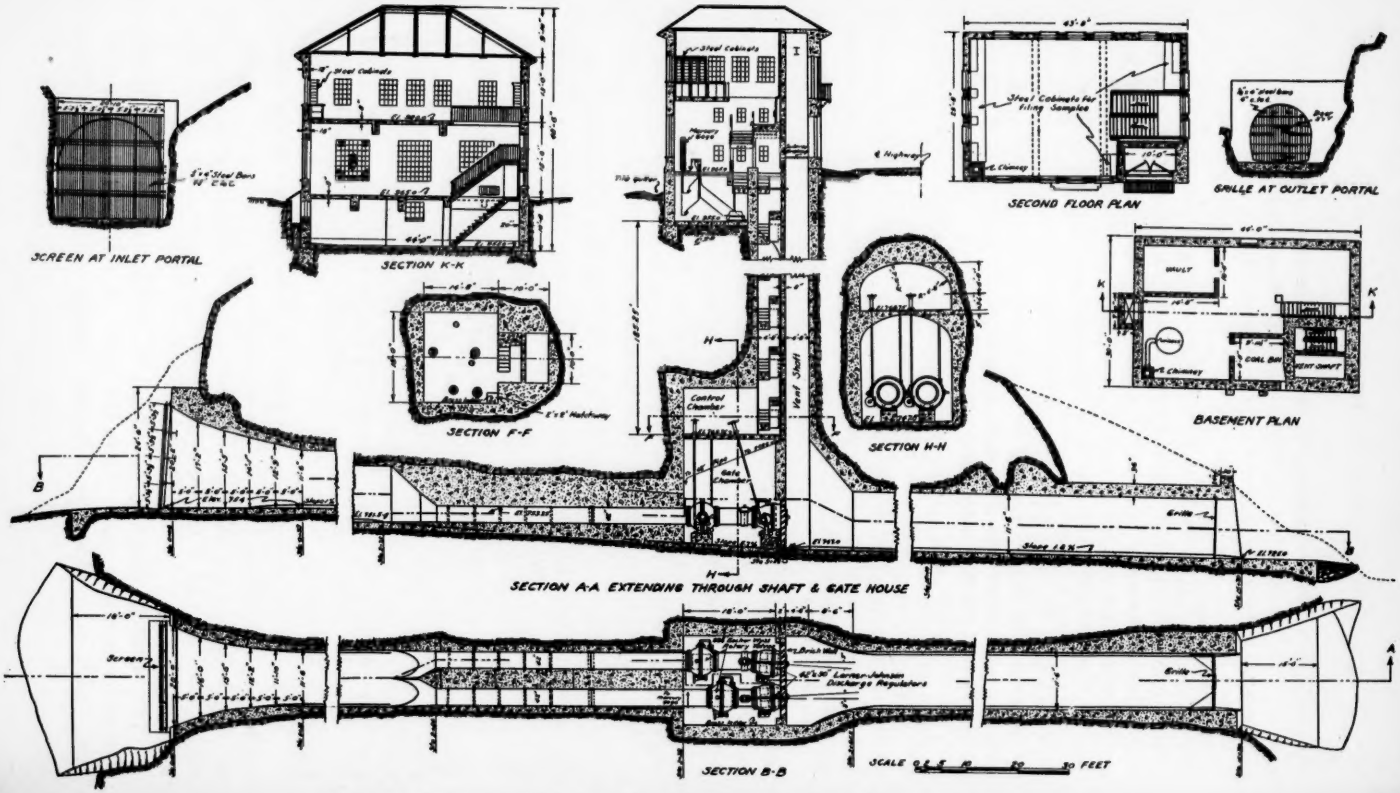

FIG. 6.— SECTIONAL PLANS AND ELEVATIONS OF TUNNEL, SHAFT AND GATEHOUSE.

were grouted under about 100 lb. per sq. in. pressure as the work progressed. Each grout pipe was plugged at the next construction joint.

In order to prevent rubbish from getting into the pipes and gates, a trash rack was installed in front of the tunnel entrance-portal after the plugging was completed. The trash rack was made of ¾-in. by 4-in. vertical bars spaced 4¾ in. center to center and supported by three horizontal 24-in.

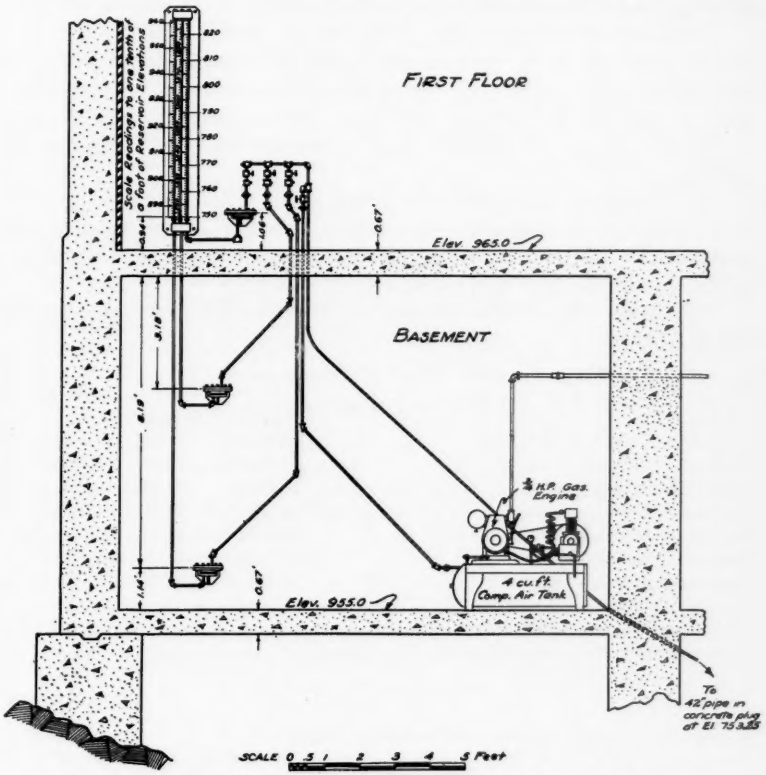
FIRST FLOOR
BASEMENT
Elev. 965.0
Elev. 955.0
¾ H.P. Gas. Engine
4 cu. ft. Comp. Air Tank
To 42" pipe in concrete plug at El 753.25
SCALE 0 3 1 2 3 4 5 Feet

FIG. 7.— MERCURY GAGE RECORDING RESERVOIR FLOW-LINE ELEVATION.

I-beams anchored to the concrete of the portal and spaced about 5 ft. center to center (Fig. 6).

At the outlet portal was installed a grille made of ½ in. by 4 in. steel bars spaced 6 in. on centers welded to horizontal bars of the same size spaced about 2 ft. on centers. The frame was anchored to the concrete lining of the tunnel. A door 3 ft. by 7 ft. in size and kept under lock and key, gives access to the tunnel through the grille. A detachable wooden

FIG. 6.— SECTIONAL PLANS AND ELEVATIONS OF TUNNEL, SHAFT AND GATEHOUSE.

bulkhead was fastened to the steel grille to prevent drafts in the tunnel and shaft.

Gatehouse. A two-story reinforced concrete building, with a cellar, 30 ft. by 44 ft. in plan and 48 ft. high was erected over the shaft of the diversion tunnel during the winter of 1931–1932. The first and second stories of this gatehouse are lined with Natco tiles.

The 4 ft.-8 in. by 10 ft. shaft opening (Fig. 6) in the northeast corner of the gatehouse extends to the top of the building. No access may be had to it from within the gatehouse. This shaft is provided with an ebony-asbestos door and serves the purpose of lowering materials or equipment to the bottom of the tunnel from the outside of the building. A second door about at the level of the second story is arranged to open automatically and prevent the creation of a vacuum in the tunnel and shaft when the gates are discharging water.

The gatehouse, in addition to giving access to the gates, is also used for housing the mercury gage showing the elevation of the water in the reservoir (Fig. 7), and for filing the records and samples of materials of Cobble Mountain Dam and its subsidiary structures.

Cost Data. The cost of constructing the diversion tunnel is summarized in Table 5.

TABLE 5.— COST OF CONSTRUCTION.

TUNNEL CONSTRUCTION.

Item.	Quantity.	Unit Cost $	Total cost — $
Open cut excavation — cu. yd.	902.4	10.00	9 024.00
Tunnel excavation — cu. yd.	10 288.5	19.00	195 481.50
Shaft excavation — cu. yd.	1 289.0	19.00	24 491.00
Concrete in tunnel lining — cu. yd.	3 420.5	20.00	68 410.00
Concrete in shaft — cu. yd.	590.0	20.00	11 800.00
Concrete in portals — cu. yd.	121.7	25.00	3 042.50
Grouting, force account, cost plus 10%			10 373.68
Temporary roads, lump sum			5 000.00
Railing in shaft and other extra work, force account			1,952.55
Total			329 575.23
Average cost of tunnel per linear foot, exclusive of shaft			182.48

TUNNEL PLUGGING.

Concrete — cu. yd.	285.4	20.00	5,708.00
Reinforcing steel — lb.	910	0.05	45.50
Larner-Johnson regulators (32 500 lb.)	2	4 651.40	9 302.80
Escher-Wyss rotary valves (25 610 lb.)	2	4 925.03	9 850.06
Pipe and accessories (48 413 lb.)			3 633.80
Placing valves and pipe — lb.	106 523	0.03	3 195.69
Rock grouting, force account			62.78
Testing valves, force account			109.44
Brick masonry, force account			205.67
Total			32 113.74

Trash racks at entrance portal (22 835 lb.)	1 400.00
Installation, force account	1 239.62
Total	2 639.62
Grille at outlet portal (2 615 lb.)	330.00
Installation	105.00
Wooden bulkhead, material and labor	53.00
Total	488.00

GATEHOUSE.

	Quantity.	Unit Cost.	Total Cost $
Structural steel — lb.	1 210.0	0.148	179.42
Elec. conduit, boxes, plates, covers, etc.— ft.	676.5	0.274	185.65
Furring and partition wall tile (12″x5″x1⅞″) — sq. ft.	3 371.0	1.088	3 664.49
Roof slabs, Porete and nail finish — sq. ft.	1 760.0	0.493	867.17
Hope casements for 30 windows — sq. ft.	618.0	1.395	862.24
Spanish tile (5½″x5½″) — sq. ft.	46.5	2.422	112.61
Imitation travertine treads, 25 pieces — sq. ft.	132.0	2.786	367.82
Doors, Ebony-Asbestos, 5 (wt. 3 200 lb.) — sq. ft.	196.0	5.051	990.04
Hardware for doors			338.80
Ceiling painting — sq. ft.	2 200.0	0.056	122.60
Earth excavation — cu. yd.	528.8	1.50	793.20
Rock excavation — cu. yd.	46.0	2.50	115.00
Old concrete excavation, force account			34.96
Concrete — cu. yd.	395.5	30.00	11 865.00
Concrete, sills, cornice, entrance — cu. yd.	15.25	60.00	915.00
Reinforcing steel — lb.	24 146.0	0.05	1 207.30
Chimney 29″x29″, 12″ flue and concrete cap — ft.	50.0	2.50	125.00
Steel roof trusses, complete — lb.	10 300.0	0.061	627.00
Roof tile, Ludowici, complete — sq. ft.	1 911.0	0.791	1 511.19
Furnace, sunbeam #5052B in place			195.02
Glass in place, complete — sq. ft.	476.0	0.743	353.85
Ornamental iron in place — lb.	3 500.0	0.317	1 111.76
Steel cabinets for filing samples			985.90
			27 531.02

Excavation of the tunnel started on August 22, 1927, and the water was diverted through the completed tunnel on July 24, 1928. The tunnel was plugged and the gates closed on August 25, 1931. The gatehouse was finished early in 1932.

The work was done by the Board of Water Commissioners of the City of Springfield with E. E. Lochridge as Chief Engineer, Hazen & Everett as Consulting Engineers, and the writer as Engineer in Charge. The members of the Board were: Edwin A. Blodgett, Chairman, 1916–1932; Dr. George L. Fenn, 1920–1931; Mayor Fordis C. Parker, ex officio, 1925–1929; Mayor Dwight R. Winter, ex officio, after 1930; James J. Shea, after 1932, Chairman, after 1933; and John M. Noonan, after 1933.

Coleman Brothers, Inc., Boston, Mass., were the contractors for the Diversion Tunnel, the Shaft and the Diversion Dam. The tunnel was plugged and the gatehouse was constructed by S. J. Groves & Sons Co. of Minneapolis, Minn., the contractor for the Cobble Mountain Dam. The actual work, however, was done by the sub-contractor, Burr M. Stark of Troy, N. Y.

NEW ENGLAND'S CLASSIC CONTRIBUTIONS TO HYDRAULIC SCIENCE

BY CHARLES W. SHERMAN.*

[*Received April 29, 1933.*]

A little consideration of the development of hydraulic science and water-works engineering, particularly in America, results in a conviction that an abnormally large proportion of the advance made during the nineteenth century was accomplished in New England. Before about 1800, comparatively little was known about hydraulics, and nearly all the experimental and theoretical research had been done in Italy, France and Germany. What little hydraulic work had been done in this country related to crude water-power developments, canals for transportation, and a very small amount of water-supply work. Probably no more scientific investigation accompanied it than such, for instance, as George Washington was able to apply to the canal in which he was so intensely interested.

In the field of experimental and theoretical hydraulics, the accomplishments of Francis, Mills, Fteley and Stearns, Ellis, Freeman, Herschel, FitzGerald, and Hazen account for much of the advance made in this country. In the development and testing of hydraulic turbines, the names of Francis, Boyden, Emerson and Herschel are particularly prominent. In matters relating to sources of water supply, their sufficiency and development, the names of Baldwin, Chesbrough, Davis, Fteley, Stearns and FitzGerald are outstanding. With regard to the quality of water and its improvement by filtration or other processes, accomplishments of Mills, Nichols, Drown, Sedgwick, Stearns, FitzGerald, E. B. Weston, Hazen and Whipple immediately come to mind. E. D. Leavitt had a prominent place in the development of water-works pumping-engines, and Dexter Brackett in matters relating to the consumption of water and restriction of waste. In the design and construction of water-works systems, Fanning, Brackett, Freeman and Coffin take primary place.

An engineer whose name is less widely known perhaps than that of most of those previously mentioned, but who deserves greater recognition than has been given him, was Charles S. Storrow, whose little book "A Treatise on Water Works for Conveying and Distributing Supplies of Water," published in Boston in 1835, first made known to American engineers the researches and formulas of French experimenters. Possibly the complicated appearance of many of the formulas quoted by Mr. Storrow and the free use which he made of calculus may have deterred American engineers from giving to this book the attention which it deserved.

*Of Metcalf and Eddy, Consulting Engineers, Boston, Mass.

Brief biographical sketches of some of the most important contributors to hydraulic science in New England follow.

Loammi Baldwin (1780–1838) was described in 1885 by Professor George L. Vose as better entitled than any other man to be called the "Father of Civil Engineering in America." He was a son of Colonel Loammi Baldwin (1745–1807) who served in the Continental Army under Washington, and later devoted most of his energies to the construction and operation of the Middlesex Canal. Loammi Baldwin, 2nd, therefore, had ample opportunity to become interested in scientific matters. He graduated from Harvard in 1800, but although he had already displayed considerable mechanical ability, he studied law and opened an office as a lawyer in Cambridge. He continued to practice law only three years, going to England in 1807 to study public works; and on his return he opened an office in Charlestown, as a civil engineer. He went abroad again in 1824 and spent a year in study — also commencing the accumulation of an engineering library.

Like other engineers of the time, he practiced in all branches of the profession, including the design and construction of fortifications, canals, railroads, mill dams, bridges, docks and buildings. He also trained in his office a number of men who afterwards rose to eminence in the profession. His practice extended throughout the developed portion of the United States, from Georgia to Maine.

The engineering structures built by Mr. Baldwin which are best known are the naval dry-docks at the Boston and Norfolk navy yards. These are splendid pieces of cut stone construction, structurally as sound today as when they were built, a century ago. Equipment for engineering construction work was then very crude if not entirely lacking. It is strange to learn that the pile-drivers at the Charlestown dry-dock were operated by treadmills.

There can be little doubt that Mr. Baldwin was considered the leading engineer of his time in America. Therefore, it is not surprising that the committee charged with studying the possibilities of a water supply for the city of Boston in 1834 should have enlisted his assistance. The amazing thing is the remarkable report which Mr. Baldwin prepared in less than five months (City Document No. 12 of 1834). That any engineer at that date should have been able to make such a report is nothing less than marvelous. This report must have served as a treatise on water supply to those having access to it. Some extracts indicating its scope are sufficient to show that Baldwin deserves high rank as a water-supply engineer.

He says: "There are four methods by which water is usually procured by the citizens of populous towns. First, by collecting in cisterns, rain water falling on roofs of houses, etc. Second, by raising it from wells made in the common way. Third, by boring into the earth and tapping springs below. Fourth, by conducting it into towns from high and distant sources, either by aqueduct, conduit pipes, or pumps." He then describes each

mode; six pages being required for the account of Artesian or Bored Wells, and twenty-six pages for Aqueducts. In the latter he describes the aqueducts of ancient and modern Rome, giving the following table of the former.

Name.		Era.	Length.	Cu. ft. per day.	Gallons per day.
1. Appian Aqueduct	B. C.	312	10.3250	3 706 575	27,724 181
2. Old Anio Aqueduct	B. C.	273	36.6775	8 932 338	66 813 887
3. Marcian Aqueduct	B. C.	146	56.9417	9 525 390	71 249 917
4. Tepulan Aqueduct	B. C.	127	14.2341	903 795	6 760 386
5. Julian Aqueduct	B. C.	35	.....	2 449 386	18 321 407
6. Virgin Aqueduct	B. C.	22	14.3116	5 085 624	38 040 467
7. Alsietina Aqueduct	A. D.	14	20.4526	796 152	5 656 016
8. Claudian Aqueduct	A. D.	49	42.1989	9 356 817	96 988 991
9. New Anio Aqueduct	A. D.	90	54.1644	9 622 878	71 979 127
	Total		249.3058	50 378 955	379 834 379

He describes also the aqueducts of Constantinople, Lyons and London; accounts of the water supplies of Edinburgh, Greenock, Glasgow, Paris, Beziers, Philadelphia, Cincinnati and Richmond, follow.

He then comes to the subject of the supply of the city of Boston, and proceeds to the investigation of the means of supplying, or of bringing within the control of the city, 5 000 000 gallons daily. He states that there are many ponds within the distance of about twenty miles, from which a supply of pure water may be had, by its natural flow to ground within four or five miles of Boston, sufficiently elevated to deliver water through pipes without the intervention of machinery, to the highest points of the city; and even to flow the floor of the State House.

Some of these Ponds were examined, and put down in the following table.

Name of Pond.	Town.	Area.			Ft. above marsh.	Distance.			Estimated daily capacity in gallons.
		A	Q	R		M	Q	R	
1. Spot Pond	Stoneham	260			143.58				1 600 000
2. Waltham Pond	Waltham	52	0	51	192.67	11	3	35	
3. Sandy Pond	Lincoln	152	1	24	222.95	16	3	26	
4. Baptist Pond	Newton	33	2	24	137.46	9	3	40	
5. Punkapog Pond	Canton	217			147.77	15	0	41	
6. Charles River	Watertown								
7. Massapog Pond	Sharon								
8. Long Pond	Natick	600	2	24	127.91	24	3	08	16 156 800
9. Farm Pond	Framingham	193			149.37	26	2	60	555 794
10. Shakum Pond	Framingham	89	2		155.01	27	0	20	
11. Learnard's Pond	Framingham	36			158.32	27	1	70	
12. Dug Pond	Natick	30?			133.66	24	0	63	
13. Morse's Pond	Needham	20?			112.40	20	0	70	
14. Bullard's Pond	Needham	35			104.45	19	0	07	

He then describes each pond separately, and adds: "From all the sources I have examined in the vicinity of Boston, as before stated, the most eligible are those of Farm and Shakum Ponds in Framingham, together with incidental ones dependent upon them; and Long Pond in Natick.*

"The best mode of bringing the water to town is by an aqueduct, without the use of pipes, to the nearest point of sufficient height to allow it to flow through cast-iron pipes to the highest land in the city."

He then gives four plans for constructing an aqueduct. "First, an open canal or drain, like common navigable canals, but on a small scale. Second, to build stone walls four or five feet high, instead of leaving the sides of the aqueduct or canal of natural earth. Third, a drain with stone walls, two or three feet apart, laid upon each side, without mortar or cement, three or four feet high, with flat stones to cover the top, and earth laid over the whole, so as effectually to conceal the work from sight, protect it from mischief and frost, and leave the ground free for ordinary use.

"The Fourth is that furnished by ancient Roman works and is like the third in form, but built in regular masonry, laid in hydraulic cement or in common mortar, and lined with cement. In this, the bottom should be stone, the top covered with the same, and the whole laid underground, or where the foundation is too low, the work to be surrounded and covered with an embankment." The latter is the one that he recommends.

He then gives his idea of the route to be taken, with a description of each section, and the cost of the whole, which he estimates at $750 000, not including the distribution in the city, as that must depend on the quantity distributed. If Long Pond should be adopted instead of Farm and Shakum Ponds, it will add, in his opinion, but $20 000 or $30 000 more.

(The foregoing has been condensed from an abstract of the original report which appeared in "History of the Water Works, Boston, 1868.")

Charles Storer Storrow (1809–1904) was a graduate of Harvard in the class of 1829. Before graduation, he had decided upon an engineering career, and during his senior year in college and for some months after, he read diligently in the library of Loammi Baldwin. He then went to Paris and studied for more than two years at the Ecole Polytechnique and the Ecole des Ponts et Chaussées. Returning to America in 1832, he became assistant engineer to Loammi Baldwin, and shortly thereafter he wrote "A Treatise on Water Works for Conveying and Distributing Supplies of Water; with Tables and Examples" (published in Boston, 1835). This "was the first treatise on hydraulics in the English language. It made available for the first time to English speaking engineers the work of the French writers, Prony, Belanger and others."

While assistant engineer to Mr. Baldwin, Mr. Storrow was engaged on the construction of the Boston & Lowell Railroad, the first steam railroad in New England. Later he was made its general manager, a position which he held until 1844. During this period he made measurements of the water used for power at Lowell, and later was influential in having James B. Francis appointed to continue the work.

In 1844, Mr. Storrow was appointed engineer of the Essex Company,

*This pond, later called Lake Cochituate, is the one from which the supply was finally taken, in 1848.

which was about to undertake the development of the water power at Lawrence. "In its development, this power stands alone of all the early American water powers as showing a clear appreciation of the end in the beginning." In 1846, when the Lawrence Scientific School of Harvard University was founded by Abbot Lawrence (largely as a result of Mr. Storrow's statements to him about engineering schools of Europe), the professorship of engineering was offered to Mr. Storrow, who declined it. Later he was a member of a state commission to investigate the condition of the Sudbury and Concord rivers; and afterwards he was sent by the State to Europe to investigate the use of compressed air in tunnelling, with a view to accelerating work on the Hoosac Tunnel. Most of his time, until he gave up active business, was devoted to the Essex Company, of which he was Treasurer and General Manager.

Mr. Storrow was an Honorary Member of the American Society of Civil Engineers.

James Bicheno Francis (1815–1892) was born in England. He came to America in 1833. A year later he was in the machine shop of the Proprietors of Locks and Canals on Merrimack River, and in 1837, on the recommendation of Mr. Storrow, he became engineer of the company. In 1845 he was appointed "Agent" (General Manager) as well, and held both positions until 1885, after which date he continued to serve as consulting engineer to the company until his death. From 1850 to 1890 he probably ranked as the most eminent hydraulic engineer in America.

The publication of his "Lowell Hydraulic Experiments" in 1855 brought to the attention of engineers everywhere the remarkable experimental work which Mr. Francis had carried on. In particular, his formula for flow over weirs, $Q = 3.33\ L\ h^{3/2}$, which is still accepted as a standard formula notwithstanding a great deal of later experimentation and the presentation of other formulas, made his name familiar to everyone doing hydraulic work.

In addition to the work reported in Lowell Hydraulic Experiments, Mr. Francis made important contributions through papers, discussions and reports. In particular, attention should be called to papers on Experiments on Flow of Water over Submerged Weirs,[1] Experiments on the Humphrey Turbine Water Wheel at the Tremont and Suffolk Mill in Lowell, Mass.,[2] On the Cause of the Maximum Velocity of Water Flowing in Open Channels being below the Surface,[3] Distribution of Rainfall during the Great Storm of October 3–4, 1869,[4] High Walls or Dams to Resist the Pressure of Water,[5] Report of the Committee on the Cause of the Failure of the South Fork Dam[6] (Johnstown, Pa.), also to Report of the Commission on the Prevention of Floods in the Valley of Stony Brook[7] (with Eliot C.

[1]Trans Am. Soc. C. E., Vol. 13.
[2]Ibid., Vol. 13.
[3]Ibid., Vol. 7.
[4]Ibid., Vol. 7.
[5]Ibid., Vol. 19.
[6]Ibid., Vol. 24.
[7]Boston City Document, 1886.

Clarke and Clemens Herschel). Mr. Francis was also prominently identified with the development and rating of the turbine water wheel. His influence upon the development of hydraulic science in America may doubtless be stated to have far exceeded that of any other man.

Mr. Francis was president of the Boston Society of Civil Engineers in 1874, and of the American Society of Civil Engineers in 1881. He was made an Honorary Member of the latter society in 1892.

Uriah Atherton Boyden (1804–1879) is known principally as the inventor of a number of improvements in turbine water wheels and of the hook gage. In his early life, he was a leather splitter and had a shop in Cambridge. Later he became interested in scientific subjects, and was a student in the office of Loammi Baldwin, in Charlestown. In 1838 he was engineer of the Nashua & Lowell Railroad, but after about 1840 devoted himself mainly to hydraulics. He accumulated a "considerable fortune" from the sale of his patents. On his death, he bequeathed about $230 000 to Harvard College, which was used in establishing an astronomical observatory in Peru.

Ellis Sylvester Chesbrough (1813–1886) did a substantial amount of his important hydraulic work for the city of Boston, although his later professional reputation was more as an expert in sewerage and sewage disposal, based largely on his work in Chicago and as consultant for many of the largest cities in this country. He was born in Maryland and had but little schooling, and his early work in engineering (1828–1846) was mainly on railroad construction. In 1846, he was chosen as engineer for the Western Division of the Boston Water Works, and had charge of the design and construction of the works for obtaining water from Lake Cochituate. In 1851, the Cochituate Water Board elected him as City Engineer of Boston, an office which he filled until 1855. (The principal, if not the only function of the City Engineer at that time, appears to have been to administer the Water Works.) During this period, he made extensive investigations and valuable reports upon the extent and effect of tuberculation in cast-iron pipes, bringing together a mass of information which appears to include a large part of what is known on this subject at the present time.

In 1855, Mr. Chesbrough went to Chicago as Engineer of Sewers, and later as City Engineer. He soon became recognized as the leading sanitary engineer in America. He was responsible for the construction of the first water-works tunnel intakes at Chicago. He was President of the American Society of Civil Engineers in 1878.

Joel Herbert Shedd (1834–1915) should be remembered especially because he more than any other engineer of recent times, maintained an office which served as a training school for young engineers. He opened an office in Boston in 1856, specializing in hydraulic and sanitary engineering. Later he removed to Providence, where he was City Engineer for a long time, designing and building considerable parts of the water and sewerage works for that city.

Joseph Phineas Davis (1837–1917) was a graduate of Rensselaer Polytechnic Institute in the class of 1856. After working on the Brooklyn Water Works for a few years, he went to Peru where he spent about five years, making surveys, plans and reports for water supplies, bridges, railroads, sewerage systems and other public works. Later he was principal assistant engineer on the construction of the St. Louis Water Works. In 1870, he became Chief Engineer of the Lowell (Mass.) Water Works and had charge of the design and construction of the system.

In 1871 he was appointed Chief Engineer of the Cochituate (Boston) Water Board, and after an investigation of all available sources, recommended the Sudbury River for an additional supply for Boston. The works for original development were carried out under his direction as City Engineer, a position to which he was elected in 1872, Alphonse Fteley being resident engineer of the Water Works. During this period, Mr. Davis also planned and began the construction of the Boston Main Drainage System.

Most of Mr. Davis' later work was for the Telephone Company and included the design and construction of systems of ducts for underground wires, studies of demand, and the like. However, he served as consulting engineer to the Croton Aqueduct Commission, the Massachusetts State Board of Health and the Metropolitan Water Board.

Mr. Davis' published contributions to hydraulic and water supply engineering are found in his reports on the Boston water and sewerage works. Many of them are not widely known. A most important part of his work was in inspiring and developing the men who worked under him, several of whom became eminent.

Hiram Francis Mills (1836–1921) was a graduate of Rensselaer Polytechnic Institute in 1856. For several years thereafter he worked under such men as Kirkwood, Worthen, Storrow and Francis. In 1869, he became Chief Engineer of the Essex Company, which controlled the water power on the Merrimac River at Lawrence. Here he made a great many hydraulic experiments, especially upon the flow of water in open channels and the distribution of velocities in channels. In 1878, he contributed to the American Academy of Arts and Sciences a paper on Piezometers which was of especial importance. The results of most of his experiments, however, were not published until after his death.

Mr. Mills' reputation was based more upon his service as a member of the Massachusetts State Board of Health than upon his more strictly engineering work. Although he received no remuneration for his services, he devoted a great deal of his time to its work, especially that relating to water supply and sewerage. Under his direction, the Lawrence Experiment Station was started and maintained; and the two classic special reports of the Board entitled "Purification of Sewage and Water" and "Examination of Water Supplies" were prepared. With Frederic P. Stearns as Chief Engineer of the Board, Thomas M. Drown as Chemist,

William T. Sedgwick as Biologist and Allen Hazen in charge of the Lawrence Experiment Station, work was accomplished which served as the foundation for most of the recent progress in improving the quality of water and sewage effluents. Having these experimental data, Mr. Mills designed and constructed (in 1892) the slow sand-filter of the Lawrence Water Works, which resulted in a very large reduction of the typhoid death rate in that city.

Mr. Mills was an Honorary Member of the American Society of Civil Engineers.

Theodore G. Ellis (1830–1883) received his early professional training in an office in Boston. In 1854, he went to Connecticut on railroad work, and later to Mexico on mining work. Early in 1861, he was engaged on surveys of Boston Harbor. Later he enlisted in a Connecticut regiment, and served through the Civil War, being brevetted a Brigadier General. After the war, he opened an engineering office in Hartford, where he spent the remainder of his life. He made extensive studies of the Connecticut River for the U. S. Government, and also conducted some experiments on the discharge of orifices, reported in a paper "Description and Results of Hydraulic Experiments with Large Apertures at Holyoke, Mass. in 1874."*

Alphonse Fteley (1837–1903) was born and educated in France. He came to America in 1865 and was located in New York until 1873, when Joseph P. Davis, City Engineer of Boston, engaged him as resident engineer in charge (under Mr. Davis) of the design and construction of the works for an additional supply of water from Sudbury River. He prepared a very complete report on this work which was published by the city under the title "Additional Supply of Water, 1883." During the progress of the work, Mr. Fteley, and his assistant, Frederic P. Stearns, carried out many experiments on the flow of water over weirs and in conduits, which were reported in a paper "Description of Some Experiments on the Flow of Water made during the Construction of Works for Conveying the Water of Sudbury River to Boston."† The Fteley and Stearns formula for flow over weirs, resulting from these experiments, is almost as well known as that of Francis.

In 1884, Mr. Fteley went to New York as principal assistant engineer of the Croton Aqueduct Commission, of which he later became Chief Engineer, having charge of the design and construction of the works constructed between 1885 and 1889.

Mr. Fteley was president of the American Society of Civil Engineers in 1898.

John Thomas Fanning (1837–1911) served through the Civil War, attaining the rank of Lieutenant Colonel. After the war, he opened an office in Norwich, Conn. He specialized in water-works engineering and designed and constructed many important works. His most notable

*Trans. Am. Soc. C. E., Vol. 5.
†Trans. Soc. C. E., Vol. 12.

achievement, however, was his book "A Practical Treatise on Hydraulic and Water Supply Engineering," published in 1877, which was the water-works man's encyclopedia for the next twenty years. In 1885, he removed to Minneapolis, where he spent the remainder of his life.

Clemens Herschel (1842–1930) was born in Boston, graduated at the Lawrence Scientific School of Harvard University (1860) and afterwards studied in France and Germany. After serving in various capacities, part of the time in the office of the City Engineer of Boston, and largely on bridge work, he became hydraulic engineer of the Holyoke Water Power Company in 1879, and constructed the famous Holyoke Testing Flume, initiating the long series of tests of hydraulic turbines carried on there. This was in succession to a commercial testing flume previously maintained for some years by James Emerson. Later (1889) he became engineer and superintendent of the East Jersey Water Company, and after 1900 was in consulting practice. He is well known as the inventor of the Venturi meter. His books "Frontinus and the Water Supply of the City of Rome" and "115 Experiments" are too well known to require description.

Mr. Herschel was President of the American Society of Civil Engineers in 1916.

Desmond FitzGerald (1841–1926) was born in Nassau, Bahama Is., but removed to Providence when quite young. On his mother's side, he was a descendant of Roger Williams.

After graduating from Phillips Andover Academy, he entered an engineer's office in Providence and soon after engaged in railroad work in Indiana and later on the Boston and Worcester Railroad. In 1872, he was made superintendent of the Western Division of the Boston Water Works. He was a pioneer in starting sanitary surveys and regular sanitary inspections of drainage areas from which water supplies were obtained. He inaugurated and pressed to a successful conclusion the first suit to prevent the pollution of waters used for public supplies; he established the first biological laboratory in connection with a water-works system and put George C. Whipple in charge. He designed and established in 1878 the recording rain gage at Chestnut Hill Reservoir. He conducted a long series of experiments on Evaporation at Chestnut Hill Reservoir and wrote a paper[1] on this subject which is still of marked importance. Other important papers were Maximum Rates of Rainfall,[2] Rainfall, Flow of Streams and Storage,[3] The Temperature of Lakes,[4] Flow of Water in 48-in. Pipes,[5] The Influence of Elevation and Wind upon Rain-gage Records,[6] Rainfall Available for Water Supply,[7] and Capacity of the Sudbury River and Lake Cochituate Reservoir in Time of Drought.[8]

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[1]Trans. Am. Soc. C. E., Vol. 15.
[2]Ibid., Vol. 21.
[3]Ibid., Vol. 27.
[4]Ibid., Vol. 34.
[5]Ibid., Vol. 35.
[6]Jour. Assn. Eng. Socs., Vol. 3.
[7]Jour. N. E. W. W. Assn., Vol. 1.
[8]Jour. N. E. W. W. Assn., Vol. 2.

Mr. FitzGerald had been President of the American Society of Civil Engineers, the Boston Society of Civil Engineers and the New England Water Works Association, and was an Honorary Member of each. He also established the FitzGerald Medal of the Boston Society of Civil Engineers.

Edmund Brownell Weston (1850–1916) began his engineering work as a student in the office of J. Herbert Shedd, then Chief Engineer of the Providence Water Works, and remained on these works until 1897. He is best known for his paper "The Results of Investigations Relative to Formulas for the Flow of Water in Pipes"[1] and his "Tables showing the Loss of Head due to Friction of Water in Pipes," originally printed in reports of the City Engineer of Providence, and later published (1896) by Van Nostrand; and his "Report of the Results obtained with Experimental Filters at the Pettaconset Pumping Station of the Providence Water Works."[2] These experiments (on mechanical filters) antedate the better known experiments at Louisville.

After 1897, Mr. Weston was President of the Jewell Export Filter Company.

Frederic Pike Stearns (1851–1919) entered the office of the city surveyor of Boston in 1868. From 1872 to 1880 he worked under Alphonse Fteley on the Sudbury River water-supply works for Boston, and collaborated with him in the hydraulic experiments which have already been mentioned. He also prepared a paper On the Current Meter, together with a Reason why the Maximum Velocity of Water flowing in Open Channels is below the Surface.[3] From 1880 to 1886 he was engaged principally on the Boston Main Drainage work. In 1886 he became Chief Engineer of the Massachusetts State Board of Health. His paper "Suggestions as to the Selection of Sources of Water Supply" appeared in the 1890 report of the Board and was reprinted in the Journal of the New England Water Works Association.[4] In 1895 he prepared the report on Metropolitan Water Supply, and later became Chief Engineer of the Metropolitan Water Board, a position which he held until 1907.

Mr. Stearns had been President of the Boston Society of Civil Engineers and of the American Society of Civil Engineers and was an Honorary Member of the New England Water Works Association.

John Ripley Freeman (1855–1932) was graduated at the Massachusetts Institute of Technology in 1876. Thereafter he was assistant engineer to Hiram F. Mills at Lawrence, and then engineer of the Associated Factory Mutual Insurance Companies. His papers Experiments relating to the Hydraulics of Fire Streams,[5] The Nozzle as an Accurate Water Meter[6] and The Arrangements of Hydrants and Water Pipes for the Protection of a

[1] Trans. Am. Soc. C. E., Vol. 22.
[2] 17th Annual Report of the Rhode Island Board of Health, 1893.
[3] Trans. Am. C. E., Vol. 12.
[4] Vol. 6.
[5] Trans. Am. Soc. C. E., Vol. 21.
[6] Ibid., Vol. 24.

City against Fire[1] should be especially mentioned in any list of hydraulic and water-supply classics of the 19th century.

Mr. Freeman had been president of the Boston Society of Civil Engineers, the American Society of Mechanical Engineers, and the American Society of Civil Engineers, and was an Honorary Member of each of them, as well as of the New England Water Works Association.

[1]Jour. N. E. Water Works Assn., Vol. 7.

THE GREAT STORM OF SEPTEMBER 16 AND 17, 1932.

BY GEORGE V. WHITE.*

[*Received February 25, 1933.*]

Following the severe storm of November 3 and 4 in 1927, which caused a great amount of damage, destruction and the loss of several lives, many papers were published describing it as well as previous great storms recorded in New England. Among the papers were: Rainfall in New England During the Storm of November 3–4, 1927, by Mr. X. H. Goodnough[1]; The Weather Map Story of the Flooding Rainstorm of New England and Adjoining Regions, November 3–4, 1927, by Weber & Brooks[2]; New England Flood of November, 1927, by H. B. Kinnison[3]. These papers described at least four great storms within the memory of our oldest people. It is the purpose of this paper to compare the storms of October 3 and 4, 1869, February 10–14, 1886, October 12–14, 1895 and November 3 and 4, 1927 with a great storm that occurred on September 16 and 17, 1932 in New England, a storm of the first magnitude which, despite the fact that it was exceeded in extent only by the storm of 1869, caused slight damage and received but little press mention.

The storm of October, 1869 appears to be the earliest storm recorded by sufficient rain gages to make possible the preparation of a satisfactory map showing lines of equal rainfall. Unfortunately, gages were scarce in Maine, and we have no record of the amount of rain that fell over nearly three-quarters of that state during this storm. From the information that is available, there appears to have been an area in New England of 13 570 sq. miles covered with six inches, or more, of rain. The center of the storm was in the vicinity of what is now the watershed for the water supply of Hartford, Conn. The highest amount of rain recorded was 12.35 in.

The storm of February, 1886 covered only southeastern New England and had its center in the vicinity of what is now the catchment area for the water supply of Providence, R. I. An area of about 3 000 sq. miles received six inches, or more, of rain. The maximum amount recorded for the storm was 9.05 in. The run-off was excessive as the ground was deeply covered with snow having an estimated water equivalent of about two inches. The temperature, low at the beginning of the storm, rose to above 50° F. before it ended, causing one of the greatest freshets recorded in southeastern Massachusetts.

Another great storm occurred in southeastern New England on October 11, 1895, lasting through October 14. About 4 070 sq. miles were

*Senior Assistant Engineer, Massachusetts Department of Public Health, Boston, Mass.

[1]This JOURNAL, Vol. XLII, No. 2 and Vol. XLIV, No. 1.

[2]*Ibid*, Vol. XLII, No. 1 and Vol. XLIV, No. 1.

[3]Contributions to the Hydrology of the United States, U. S. Geological Survey Water Supply Paper No. 636.

covered with six inches, or more, of rain. The center of this storm was somewhat northwest of the area covered by the big storm of February, 1886 and extended from Woonsocket, R. I., to Waltham, Mass. The maximum amount of rainfall recorded was 8.86 in. This storm followed a period

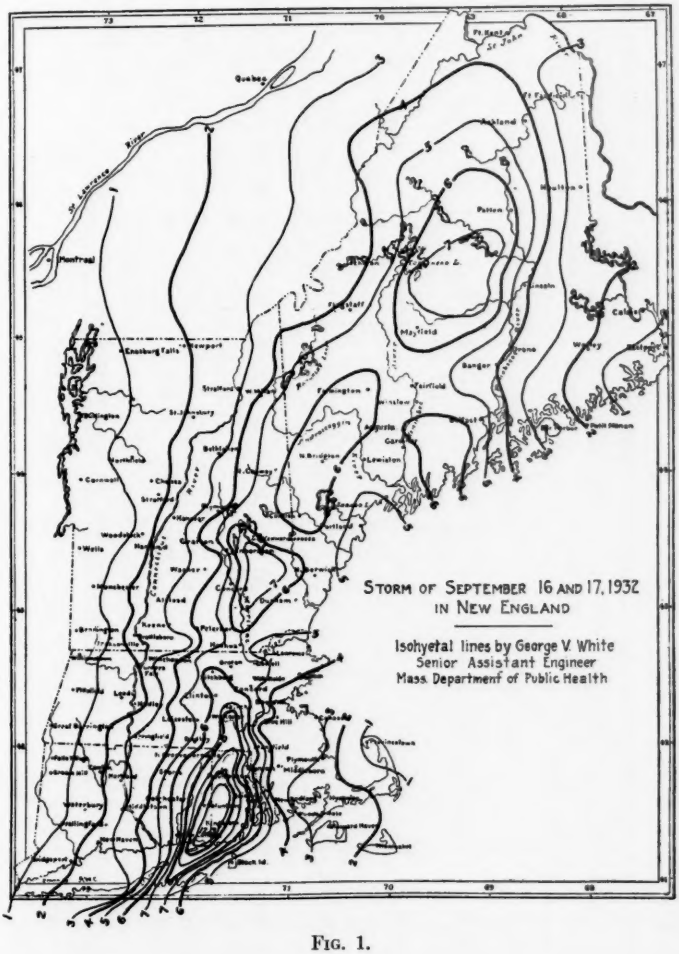
STORM OF SEPTEMBER 16 AND 17, 1932
IN NEW ENGLAND
Isohyetal lines by George V. White
Senior Assistant Engineer
Mass. Department of Public Health

FIG. 1.

of considerable drought, including the very dry years of 1894 and 1895 up to October, and its effects were only slight, filling the streams and flooding grounds that are generally inundated during high flows.

The storm of November 3 and 4, 1927 covered all New England except along the sea coast and northern Maine. Five separate areas, aggregating

TABLE 1.— ACCUMULATIVE NORMAL AND ACTUAL RAINFALL IN NEW ENGLAND FROM JUNE 1, 1929 TO OCTOBER 1, 1932.*

	Maine.		New Hampshire.		Vermont.		Massachusetts.		Rhode Island.		Connecticut.	
	Nor.†	Act.‡	Nor.	Act.	Nor.	Act.	Nor.	Act.	Nor.	Act.	Nor.	Act.
June, 1929 — May, 1930	39.54	35.06	38.30	33.99	33.11	34.59	44.42	32.77	45.13	33.83	45.72	32.85
Excess or Deficiency.	—4.48		—4.31		+1.48		—11.65		—11.30		—12.87	
June, 1929 — May, 1931	79.08	68.28	76.60	66.82	66.22	67.57	88.76	72.28	90.26	70.68	91.44	71.99
Excess or Deficiency.	—10.80		—9.78		+1.35		—16.48		—19.58		—19.45	
June, 1929 — May, 1932	118.62	109.50	114.90	107.38	99.33	107.44	133.02	114.14	135.39	110.16	137.16	110.79
Excess or Deficiency.	—9.12		—7.52		+8.11		—18.88		—25.23		—26.37	
June, 1929 — August, 1932	129.49	120.02	125.50	118.27	109.48	118.52	144.28	123.29	146.26	121.14	149.19	123.19
Excess or Deficiency.	—9.47		—7.23		+9.04		—20.99		—25.12		—26.00	
June, 1929 — September, 1932	133.13	125.73	128.81	124.35	112.51	121.59	147.73	130.05	149.61	130.42	152.73	127.66
Excess or Deficiency.	—7.40		—4.46		+9.08		—17.68		—19.19		—25.07	

*The actual figures are for one station in each state except for Massachusetts where the average of eight stations is given. For Maine, the Rumford Falls record is used; for New Hampshire, Concord; for Vermont, Northfield; for Rhode Island, the Providence Weather Bureau; for Connecticut, New Haven.

†Normal rainfall — in inches.

‡Actual rainfall experienced — in inches.

8 474 sq. miles, received six inches, or more, of rain. There were two high areas in this storm; one over the Green Mountains of Vermont, the other extending from Westerly, R. I. to Southboro, Mass., the immediate vicinity of the high of the storms of October, 1895 and February, 1886. The maxi-

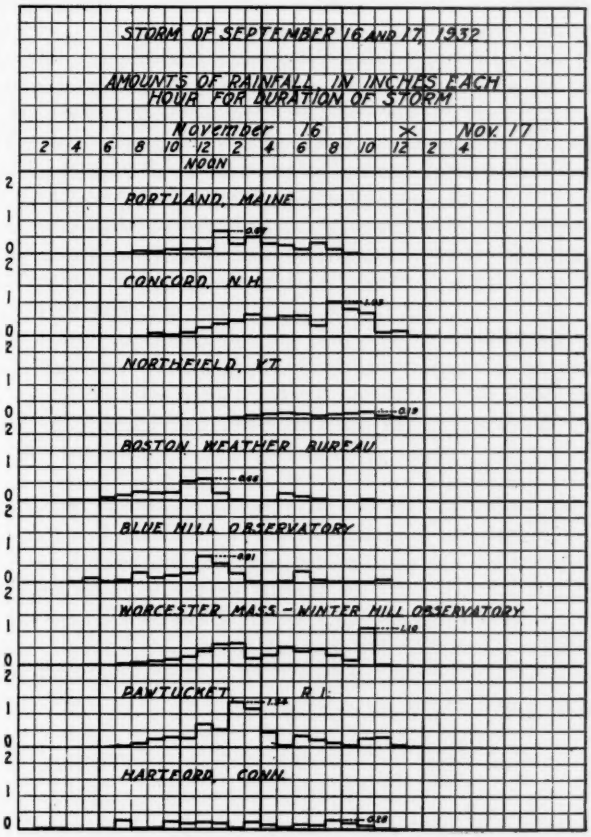
STORM OF SEPTEMBER 16 AND 17, 1932
AMOUNTS OF RAINFALL IN INCHES EACH HOUR FOR DURATION OF STORM
NOVEMBER 16
NOV. 17
2 4 6 8 10 12 2 4 6 8 10 12 2 4
NOON
PORTLAND, MAINE
CONCORD, N.H.
NORTHFIELD, VT.
BOSTON WEATHER BUREAU
BLUE HILL OBSERVATORY
WORCESTER MASS. - WINTER HILL OBSERVATORY
PAWTUCKET, R.I.
HARTFORD, CONN.

FIG. 2.

mum amount recorded in the Vermont high area was 9.45 in. The secondary high was 9.37 in. This storm followed a very wet period; the ground was well saturated, and ponds and streams, except in a few cases, were filled and overflowing. There resulted freshets and destruction such as had never before been known in old New England.

On September 16 and 17, 1932, New England experienced a storm surpassing in magnitude the storm of November, 1927 and all others back to the one occurring on October 3 and 4, 1869. Few probably realize the

severity of this recent storm, because it caused little or no damage and accounts of it occupied but a small amount of space in the daily papers. This storm followed a long period of excessive drought in every state of New England except Vermont. This is shown in Table 1 which gives the accumulated normal rainfall for the New England states and accumulated rainfall recorded at representative stations, together with the excess or deficiency for each twelve-month period beginning June 1, 1929, and for the thirty-nine months preceding this storm. The values in this table and the run-off figures given in Table 2 show the ground and cover to have been in a condition in which even a greater storm would not have caused much damage. Only a few representative peak run-off figures for this storm are shown in Table 2, together with the peak flows following the November, 1927 flood at the same points of measurement. The marked difference between these two sets of figures is due not primarily to rainfall, but to ground and cover conditions preceding the storms. In the case of the Wachusett watershed the rainfall on the catchment area during the storm of November, 1927 was 4.90 in.; but for the four months preceding there had been a rainfall of 8.33 in. in excess of the normal. The peak flow of the South Branch of the Nashua River was 2 680 cu. ft. per sec. During the storm of September, 1932 the rainfall on this watershed was 5.77 in., but the accumulated deficiency of rainfall extending over a period of thirty-nine months prior to September 1, 1932 was 22.43 in. The peak run-off following the 1932 storm was 1 381 cu. ft. per sec.

The figures for the Connecticut River at Sunderland reflect the fact that the 1932 rainfall on the Connecticut River watershed probably did not exceed three inches. The peak discharge of the Penobscot River following the 1932 storm is greater than for the 1927 storm due to the fact that there was not quite so severe a drought in this part of Maine previous to September, 1932 as there was in other sections of New England, and to the great amount of rain, averaging about 7.50 in., that fell on the watershed above West Enfield during the storm.

TABLE 2.— PEAK FLOWS FOLLOWING THE STORMS OF NOVEMBER, 1927 AND SEPTEMBER, 1932.

Stream	Station	Run-off — cu- ft. per sec. Nov., 1927	Sept., 1932
Blackstone	Worcester, Mass.	790	410
Blackstone	Woonsocket, R. I.	22 200	4 570
Pawtuxet	Kent, R. I.	10 900	7 400
Quinebaug	Jewett City, Conn.	12 500	5 300
Charles	Charles River Village, Mass.	755	553
Nashua, South Branch	Clinton, Mass.	2 680	1 381
Connecticut	Sunderland, Mass.	165 000	13 200
Merrimack	Franklin Junction, N. H.	67 000	14 800
Merrimack	Lawrence, Mass.	70 360	15 460
Androscoggin	Lewiston, Me.	60 000	34 800
Kennebec	Waterville, Me.	76 800	47 600
Penobscot	West Enfield, Me.	60 800	67 900

In preparing the isohyetal map shown in Figure 1, the records of 44 rainfall stations in Maine, 29 in New Hampshire, 35 in Vermont, 150 in Massachusetts, 11 in Rhode Island, 20 in Connecticut, 11 on Long Island and in eastern New York and 4 in Canada — a total of 304 — were used. These records give complete control of the location of the lines of equal rainfall throughout New England, except for a distance of about 42 miles between Block Island, R. I., and Bridgehampton, Long Island. A record from the most easterly end of Long Island would have been valuable, as it would have shown whether or not a heavy, narrow storm had passed between these points from the sea to Westerly, R. I. The map shows eight inches, or more, of rain covering a narrow strip from Long Island through Westerly and Woonsocket, R. I., to Framingham, Mass., an area of 1 555 sq. miles. The highest rainfall recorded in this area was 12.13 in. at Westerly, R. I. Mr. Barns, for over forty years a careful observer at this place, states: The storm broke at 3:00 A.M. on September 16 and rained in sheets continuously until 10:30 P.M. that night. His record shows that in 19½ hours 12.13 in. of rain fell.

The intensity of the rain during this storm is shown in Figure 2. The results presented are plotted from recording rain-gage records. When compared with similar records made during the storm of November, 1927, they show that the Worcester gage recorded the highest rate (1.42 in. per hour) for the 1927 storm, and the Pawtucket gage (1.34 in. per hour) for the 1932 storm. In sixteen hours the latter gage recorded 6.38 in. Its six-hour total was 4.45 in. Both amounts are greater than for previous storms for the same period of time.

Table 3 shows the areas in square miles in each New England state upon which a specified amount of rain fell.

Table 4 gives the total areas for similar specified amounts for the five great storms of October, 1869, February, 1886, October, 1895, November, 1927 and September, 1932.

Table 5 records the detailed reports of the different rainfall stations by states.

The following meteorological account of the storm is given by Brooks:*

This rainstorm occurred with the passage of a tropical cyclone from the Gulf of Campeachy that joined a northern low as it reached New England. When this storm crossed the Florida peninsula September 14–15 it was attended by a narrow belt of very heavy rains "ranging from 5 to 15 inches, the greatest 48-hour total, 15.15 inches, occurring at Apalachicola, where low sections of the city were flooded to a depth of about 5 feet. . . . [A night train was derailed.] . . . There was considerable damage to crops along the path of the storm by washing rains and high winds." In Georgia, 11.61 inches fell at Brunswick, September 14–15. The heaviest in South Carolina was 5.15, at Charleston. Beaufort, N. C., had 6.34 inches, September 14–16. Though the main center is shown on the weather map to have passed southeast of New England, being centered south of Nantucket

*Brooks, C. F. Bull. Am. Metl. Soc., Jan. 1933, pp. 26–27.

the evening of the 16th, the winds at Blue Hill and Boston indicated that a low pressure center passed through New England west of those stations. The wind at Blue Hill was northeasterly till 1:00 P.M., with a maximum velocity of 38 mi/hr, then SExE till 10, with a maximum of 41 mi/hr. At 10:25 the wind swung from SSE to W, then to NWxN at midnight, the velocity continuing at 25–30 mi/hr. By 3:00 A.M, however, the direction had backed to WNW and the velocity had risen to 38 mi/hr. At 4:00 A.M. it reached its maximum, 47 mi/hr. The rainfall, which at Blue Hill was 3.68 inches, ceased at 10:25 P.M. At 8:00 A.M. the temperature was 61, at 2:00 P.M. 63.8 and at 8:00 P.M., 65°F. Dense fog (cloud) enveloped the hill throughout the day.

The weather maps of the mornings of September 16 and 17, 1932, are strikingly similar to those of the mornings of November 3 and 4, 1927, except that the low centers are about 200 miles farther east and the pressures are not quite so low. There was the same strong high pressure area over the ocean to the east, and the same double-centered low with south-north axis, favoring a great flow of warm moist oceanic air into New England and its precipitation over a relatively narrow belt.

TABLE 3.—AREA IN EACH NEW ENGLAND STATE IN WHICH SPECIFIED AMOUNTS OF RAIN WERE EXCEEDED DURING THE STORM OF SEPTEMBER 16 AND 17, 1932.

Rainfall (Inches) in Excess of	AREA — SQUARE MILES.					
	Conn.	R. I.	Mass.	N. H.	Maine	Total
11	110	210	..	..	..	320
10	270	335	..	..	..	605
9	425	590	..	..	..	1 015
8	630	775	150	..	..	1 555
7	670	920	290	440	1 490	3 810
6	840	1 050	775	1 195	7 165	11 025
5	1 255	1 060	1 865	3 540	15 800	23 520
4	1 635	1 060	3 205	5 090	22 690	33 680

TABLE 4.— COMPARISON OF RAINFALL AREAS OF GREAT STORMS IN SQUARE MILES.

Rainfall (Inches) in Excess of	AREA—SQUARE MILES.				
	October, 1869.	February, 1886.	October, 1895.	November, 1927.	September, 1932.
12	40	....	....	.....	.....
11	255	....	....	.....	320
10	370	....	....	.....	605
9	902	....	....	497	1 015
8	2 264	750	590	1 795	1 555
7	7 154	1 500	2 045	4 042	3 810
6	13 572	3 000	4 070	8 474	11 035
5	.....	4 850	6 340	22 034	23 520
4	.....	7 600	....	.....	33 680

TABLE 5.— INCHES OF RAINFALL IN STORM OF SEPTEMBER 16 AND 17, 1932.

MAINE.

	Sept. 15	16	17	Total.
Mt. Agamanticus	...	5.48	....	5.48
No. Bridgton	...	2.78	4.15	6.93
Orono	.01	1.32	3.03	4.36
Ripogenus Dam	...	7.72	.63	8.35
Rumford	.20	5.34	1.22	6.76
Farmington	.33	2.10	4.20	6.63
Gardiner	...	2.39	3.20	5.59
Lewiston	T.*	4.35	1.13	5.48
Bangor	...	1.00	3.08	4.08
Portland	...	3.96	.34	4.30
Bar Harbor	...	1.06	1.19	2.25
Eastport	...	.07	.20	.27
Ellsworth	...	1.75	.48	2.23
Greenville	.47	1.84	5.63	7.94
Lincoln	...	3.00	1.50	4.50
Millinocket	.16	5.69	1.55	7.40
Milo	.31	2.64	4.99	7.94
Aziscohos Dam	.37	1.91	1.84	4.12
Brassua Dam	...	.03	4.15	4.18
Danforth	...	1.00	1.60	2.60
E. Winthrop	...	3.04	2.73	5.77
Eustis	.19	1.01	2.55	3.75
Fort Fairfield	...	....	2.03	2.03
Fort Kent	.18	2.23	1.16	3.57
Hiram	...	2.41	2.13	4.54
Houlton	...	1.11	1.35	2.46
Jackman	.34	.56	2.19	3.09
Lincoln	...	3.00	1.50	4.50
Machias	...	....	.85	.85
Madison	.02	3.23	2.21	5.46
Middle Dam	.64	2.54	2.16	5.34
Moosehead	.15	....	5.00	5.15
Oldtown	...	1.39	2.67	4.06
Presque Isle	.04	.05	2.53	2.62
Sebec	.39	6.05	.61	7.05
The Forks	.32	....	5.39	5.71
Upper Dam	.46	....	4.45	4.91
Upton	.50	1.87	2.63	5.00
Van Buren	.44	.67	2.33	3.44
W. Rockport	...	....	6.72	6.72
Winslow	...	5.42	.07	5.49
Woodland	...	1.24	.37	1.61
Livermore	.40	....	6.27	6.67
Oquossoc	...	....	3.90	3.90

*T—Trace.

NEW HAMPSHIRE.

	Sept. 15	16	17	Total.
Twin Mountain	T.	1.00	2.70	3.70
W. Stewartstown	...	2.70	....	2.70
Lakeport	...	6.90	....	6.90
Glencliff	...	2.55	....	2.55
Berlin	.30	.01	5.15	5.46
Concord	...	5.95	.03	5.98
Keene	...	1.52	.99	2.51
Manchester	...	....	6.93	6.93
Franklin	...	2.03	1.97	4.00
Hampton	...	3.65	1.82	5.47
Plymouth	...	2.00	1.36	3.36
Durham	...	3.35	2.13	5.48
Bethlehem	.09	.87	1.33	2.29
Dixville Notch	...	3.35	....	3.35
Errol	...	1.45	2.12	3.57
Hanover	T.	1.42	.72	2.14
Lincoln	.40	0	3.60	4.00
Littleton	.10	....	2.50	2.60
Milan	.76	1.73	1.65	4.14
New Durham	...	8.00	....	8.00
Newport	...	....	2.60	2.60
Pinkham Notch	.03	4.97	1.69	6.69
1st Conn. Lake	...	3.23	....	3.23
W. Lebanon	...	2.17	.25	2.42
Wolfsboro Falls	...	....	5.75	5.75
York Pond	...	3.55	....	3.55
Fitzwilliam	...	2.37	....	2.37
Nashua	.06	5.83	....	5.89

VERMONT.

	Sept. 15	16	17	Total.
Searsburg	...	.72	.22	.94
Vernon	...	....	2.11	2.11
Somerset	...	....	.77	.77
Cavendish	...	.71	1.13	1.84
Bellows Falls	...	....	2.66	2.66
Whitingham	...	....	.66	.66
Northfield	...	1.25	T.	1.25
Burlington	.02	.07	.06	.15
St. Johnsbury	.17	.30	1.75	2.22
Brattleboro	...	T.	2.24	2.24
Bennington	...	.10	....	.10
Bloomfield	...	.04	2.74	2.78
Chelsea	...	....	1.63	1.63
E. Barnet	...	....	2.26	2.26
E. Ryegate	...	....	2.16	2.16
Enosburg Falls	...	....	.41	.41
Gilman	...	2.40	....	2.40
Guildhall	...	....	2.66	2.66
McIndoes	...	....	1.90	1.90

VERMONT (Cont'd)

	Sept. 15	16	17	Total.
Mays Mills	...	....	1.56	1.56
Newfane	...	.55	1.15	1.70
Newport	...	.59	1.23	1.82
Readsboro	...	....	0.65	.65
Rochester	...	....	.65	.65
Rutland	...	....	.34	.34
St. Albans	...	....	.15	.15
Searsburg Mt.	...	....	.72	.72
Searsburg Sta.	...	....	.75	.75
S. Londonderry	...	....	1.02	1.02
W. Burke	...	....	1.90	1.90
W. Hartford	...	....	1.83	1.83
White R. Junc.	...	....	2.42	2.42
Wilder	...	....	2.47	2.47
Wilmington	...	....	.78	.78
Woodstock	...	....	.92	.92

MASSACHUSETTS.

	Sept. 15	16	17	Total.
Clinton	...	....	6.50	6.50
Holyoke	...	.03	2.12	2.15
Jefferson	...	5.76	.02	5.78
Millbury	.15	6.13	....	6.28
Adams	...	0.48	.06	.54
Gloucester	...	3.78	.21	3.99
Shelburne Falls	...	1.47	....	1.47
Pittsfield	...	.44	....	.44
Worcester (Clarks' Univ.)	...	3.54	2.01	5.55
Weston	...	4.50	1.80	6.30
Provincetown	...	.56	....	.56
New Bedford (W. B.)	...	2.42	.12	2.54
Turners Falls	...	1.77	....	1.77
Lake Cochituate	...	6.04	.02	6.06
Framingham	...	7.00	.03	7.03
Ashland	...	7.57	T.	7.57
E. Wareham	...	.47	2.02	2.49
Stockbridge	...	.41	....	.41
Fall River	...	3.65	....	3.65
Nantucket	...	1.77	.06	1.83
Brockton	...	2.43	.50	2.93
Lawrence	...	....	4.65	4.65
Worcester (Winter Hill)	...	4.16	1.43	5.59
So. Carver	.46	2.02	....	2.48
Norton	...	....	3.38	3.38
Springfield	...	3.19	T.	3.19
Hoosac Tunnel	...	....	.52	.52
Leominster	.11	6.53	....	6.64
Williamstown	T.	.34	....	.34
Haverhill	...	2.82	1.24	4.06
Haverhill (Kenoza Lake)	.06	4.41	....	4.47

MASSACHUSETTS (Cont'd)

	Sept. 15	16	17	Total.
Amherst	...	1.94	....	1.94
Fitchburg	...	6.39	....	6.39
Lowell	...	5.55	....	5.55
Mt. Wachusett	...	....	6.64	6.64
Princeton	...	....	5.28	5.28
Sterling	...	5.41	.02	5.43
Taunton	...	3.90	....	3.90
Boylston	...	....	6.60	6.60
Marlboro	.12	7.52	....	7.64
Blue Hill Obs.	...	.18	3.68	3.88
Chestnut Hill	...	3.28	.03	3.31
Concord	...	4.90	.68	5.58
Drury	...	.57	.03	.60
Hyannis	...	2.10	.60	2.70
Middleton	.39	3.85	....	4.24
Plymouth (W. B.)	...	2.98	....	2.98
Rockport	...	3.60	....	3.60
Spot Pond	.02	3.52	.01	3.55
Swampscott	...	3.33	.18	3.51
Boston (W. B.)	...	3.10	....	3.10
Westfield (W. B.)	...	2.14	....	2.14
Cordaville	...	8.17	....	8.17
Sudbury Dam	...	7.69	....	7.69
W. Rutland	...	.05	4.59	4.64
Ashby	...	2.23	2.73	4.96
Athol	...	3.86	....	3.86
Phillipston	...	3.59	....	3.59
Athol	...	3.35	....	3.35
Attleboro	.26	6.41	....	6.67
Berlin	.10	6.85	....	6.95
Barre	...	3.38	....	3.38
White Valley	...	3.08	....	3.08
Wenham Lake	.22	4.08	.03	4.33
W. Roxbury	.31	3.12	....	3.43
Blardford	...	1.10	....	1.10
State Farm	.05	3.15	....	3.20
Brockton	.20	3.10	....	3.30
Cummington	...	.78	....	.78
Chesterfield	...	1.04	....	1.04
Chester	...	.68	....	.68
Charlton	.05	4.50	....	4.55
Colrain	...	1.75	....	1.75
Chicopee Falls	...	2.46	....	2.46
Chatham	.24	.92	....	1.16
No. Dana	...	3.14	....	3.14
So. Egremont	...	.34	T.	.34
Dana	T.	3.01	.02	3.03
Enfield	T.	2.43	T.	2.43
Franklin	.07	6.78	....	6.85
Falmouth	.87	2.38	T.	3.25
Groton	.20	5.50	....	5.70

MASSACHUSETTS (Cont'd)

	Sept. 15	16	17	Total.
Gardner	...	4.75	T.	4.75
Granville	...	1.56	....	1.56
Greenfield	...	1.87	....	1.87
Greenwich	...	2.80	....	2.80
Hubbardston	.06	4.32	.02	4.40
Hingham	...	3.70	....	3.70
Accord Pond	...	3.03	....	3.03
Heath	...	1.15	....	1.15
Hardwick	...	3.35	....	3.35
Williamsville	...	3.90	.03	3.93
Huntington	...	1.10	....	1.10
Holden (Kendall)	.06	5.70	.02	5.78
Ipswich	.12	4.24	....	4.36
Lawrence	.07	4.57	....	4.64
Littleton	.21	5.15	.02	5.38
Ludlow	...	2.27	....	2.27
Lynn	...	3.47	.01	3.48
Milford	...	6.11	1.03	7.14
Middleboro	.48	2.62	.03	3.13
Mansfield	.19	3.18	....	3.37
Monson	.16	2.71	....	2.87
Manchester	.10	3.97	....	4.07
Millis	.32	4.23	....	4.55
Middlefield	...	....	....	....
Northbridge	.08	6.40	.01	6.49
Newburyport	.20	4.30	....	4.50
Northfield	...	2.32	....	2.32
New Braintree	...	3.54	.01	3.55
Norwood	.20	3.27	....	3.47
Newton	.18	3.23	T.	3.41
New Bedford	.23	2.67	....	2.90
New Bedford (Clarks Pt.)	.51	1.73	....	2.24
No. Andover	.15	4.04	....	4.19
New Salem	...	2.97	....	2.97
W. Otis	...	.65	....	.65
Oak Bluffs	...	2.25	....	2.25
Peabody	.54	4.00	....	4.54
Plainfield	...	.97	....	.97
Peru	...	.52	....	.52
Pembroke	.32	2.47	....	2.79
Pelham	...	2.72	.01	2.73
Prescott	...	2.53	....	2.53
Bondsville	T.	1.46	....	1.46
Petersham	...	2.59	....	2.59
Wollaston	.21	2.98	....	3.19
Rutland	...	5.43	.01	5.44
No. Rutland	...	4.24	.02	4.26
Southbridge	.44	3.93	....	4.37
Shelburne Falls	...	1.44	....	1.44
Shutesbury	...	2.50	....	2.50
Salem	.38	3.74	....	4.12

MASSACHUSETTS (Cont'd)

	Sept. 15	16	17	Total.
Savoy	.56	.16	....	.72
Lakeville	...	3.19	....	3.19
Baldwinville	...	3.90	.01	3.91
Winchendon	...	3.88	....	3.88
Webster	...	5.84	....	5.84
Ware	...	2.75	....	2.75
Wrentham	.12	4.50	....	4.62
Warwick	...	3.52	....	3.52
Wendall	...	2.90	....	2.90
Warren	.03	3.04	.01	3.08
Onset	.30	2.03	....	2.33
Westfield (W. W.)	...	1.95	.05	2.00
E. Walpole	.19	3.99	....	4.18
Worthington	...	.72	....	.72
Walpole	.18	4.22	....	4.40
Williamsburg	...	1.15	....	1.15
W. Ware	...	2.54	.05	2.59
W. Brookfield	.02	3.27	....	3.29

RHODE ISLAND.

	Sept. 15	16	17	Total.
Kingston	...	6.23	2.07	8.30
Providence	...	6.17	....	6.17
Woonsocket	...	9.30	....	9.30
Westerly	...	12.13	....	12.13
Block Island	...	4.31	.02	4.33
Pawtucket	...	*	6.74	6.74
Rocky Hill	.32	8.60	....	8.92
Hopkins Mills	.66	9.12	.05	9.83
N. Scituate	.60	10.00	.03	10.63
Westcott	.68	9.20	.03	9.91
Scituate Dam	.44	11.45	....	11.89

CONNECTICUT.

	Sept. 15	16	17	Total.
E. Hartland	...	1.80	....	1.80
Falls Village	...	.52	....	.52
Bridgeport	...	1.45	.03	1.48
N. Grosvenordale	...	6.35	....	6.35
New Haven	...	2.28	T.	2.28
Cream Hill	...	0.53	....	0.53
Hartford	...	2.55	T.	2.55
Storrs	...	2.49	1.04	3.53
New London	...	4.16	1.30	5.46
Norwalk	...	1.08	....	1.08
Barkhamsted	...	....	1.21	1.21
Burlington	...	....	1.60	1.60
Colchester	...	2.35	.92	3.27

*Amount included in the following day.

CONNECTICUT (Cont'd)

	Sept. 15	16	17	Tota
Collinsville	...	T.	1.57	1.57
New Hartford	...	....	1.19	1.19
W. Hartford	...	....	1.20	1.20
Baltic	...	2.70	1.79	4.49
Waterbury	...	.79	.21	1.00
L. Konomac	...	2.51	3.20	5.71
Norwich	...	6.80	....	6.80

NEW YORK.

	Sept. 15	15	16	Total.
Bridgehampton	...	6.94	.90	7.84
Cutchogue	...	2.97	....	2.97
Setucket	...	2.48	.05	2.53
Farmingdale	...	1.77	....	1.77
Flushing	...	.93	....	.93
Mt. Vernon	...	.75	....	.75
New York City	...	.62	....	.62
Scarsdale	...	.55	....	.55
Bedford Hills	T.	.57	....	.57
Wappingers Falls	...	.45	T.	.45
Albany	T.	T.	.13	.13

DISCUSSION.

E. C. CRAIG.* Data on rainfalls and floods of great magnitude, as set forth in this paper, are always of great interest to those concerned with hydraulic problems. The rainfall run-off data obtained by the Providence Water Works since 1916 at several gaging stations located on the 92.8 square mile Scituate watershed—the source of supply—may also prove of interest in this connection.

The most recent storm of great magnitude affecting the Scituate watershed (on North Branch Pawtuxet River) occurred on September 16 and 17, 1932, when in a period of about 20 hours between about 5:00 A.M., September 16 and 1:00 A.M., September 17 an average of 10.24 in. of rainfall—none as snow or ice — was observed with five rain gages. On November 3 and 4, 1927 an average of 7.26 in. of rainfall was observed with four rain gages over a period of about 13 hours between 7:00 P.M., November 3 and 8:00 P.M. November 4.

*Superintendent, Scituate Reservoir Division, Department of Public Works, Providence, R. I.

Some of the details are shown in the following schedule:

	Rainfall — Inches. Sept. 16–17, 1932.	Nov. 3–4, 1927.
Rocky Hill	8.92*	6.86
Hopkins Mills	9.82	7.85†
North Scituate	10.63	6.80*
Westcott	9.91	.....
Scituate Dam	11.89†	7.51
Average	10.24	7.26
Estimated Duration of Storm — Hours	20	13
Max. Rate of Run-off— 1 Hour	115	132
Cu. ft. per sec. per 2 Hours	88	130
sq. mile 4 Hours	75	125

A 100-year local rain gage record for Hope Reservoir Station, Providence, indicates that the September 16–17, 1932 storm has probably not been exceeded at a single station in this locality for at least a hundred years. Considering maximum monthly rainfalls at a single gaging station, a maximum of 13.74 in. in September, 1932 at a gage on Scituate watershed compares with a maximum of 11.45 in. in November, 1927 for the 100-year record at Hope Reservoir, Providence.

Peak rates of run-off and the amount of damage resulting in the great storms referred to have shown wide differences and great variations. In the September, 1932 storm, for instance, 41 per cent. more rainfall was experienced than for the November, 1927 storm, but the peak rates of flow were only from 60 to 87 per cent. of those reached in November, 1927. In November, 1927, however, the ground was well saturated and the rate of precipitation more intensive. In September, 1932 the ground and storage conditions seem to have been ideal for absorbing water and allaying flood conditions.

The past few years have produced several high and low extremes in rainfall and run-off for the Scituate watershed that are worthy of mention. During the calendar year 1930, statistics for Scituate watershed show a rainfall of 35.58 in., a run-off of 11.82 in. or 33 per cent. collected. This compares with a 16-year average rainfall of 46.61 in., a run-off of 25.20 in. or 54.1 per cent. collected. On the other hand, between September 15 and November 15, 1932, in a period equivalent to two months, there was a rainfall of 24.30 in., and a run-off of 11.38 in. or 46.8 per cent. collected. The 24.30-in. rainfall in this period (equivalent to two months) was 52 per cent. of the 16-year average rainfall, and the run-off of 11.38 in. was 45.2 per cent. of the 16-year average run-off. The run-off from September 15 to November 15, 1932 was over 96 per cent. of the entire run-off for the dry calendar year 1930.

*Minimum.
†Maximum.
Scituate watershed has a gross area of 92.8 sq. miles.
Peak rates of flow occurred at a time when only 84 sq. mi. were contributing.

Floods of great magnitude result when a number of obvious things happen to occur at the same time. Rainfalls of much less magnitude than experienced in 1932, if occurring with reservoirs overflowing and the ground in a saturated condition, can produce peak rates of flood discharge of almost unbelievable intensity. If an average of about 10¼ in. of rainfall, such as occurred within a period of about 20 hours in September, 1932, had fallen on frozen ground or a thoroughly saturated watershed with reservoirs overflowing, it is disturbing to picture the flood conditions and damage that would have resulted.

H. B. KINNISON.* Disastrous floods in the past have been the result of unusually severe storms. The few exceptions in New England to this statement have been associated with the melting of a heavy snow cover usually accompanied by more or less rain. These spring floods are more regular and perhaps more numerous than those resulting from heavy autumn rains but they usually are not so severe or disastrous as the autumn floods. An example of a severe flood resulting from a combination of rainfall and melting snow is the one which occurred on February 8 to 10, 1886. The disastrous flood of 1927, however, was the direct result of heavy rainfall occurring from a tropical storm originating in the West Indies.

When unusually heavy precipitation concentrated in a short period of time is experienced, a severe and perhaps disastrous flood is expected to follow. While it has been realized that factors other than those directly relating to rainfall influence the run-off from such a storm, the fact has probably not been appreciated that an area may be visited by a storm of major proportions accompanied by record-breaking precipitation without the occurrence of an unusual flood. Such was the case at the time of the storm of September 16, 1932.

Beyond question this storm is the greatest experienced in the New England states since rain gages have been sufficiently well distributed to determine in a satisfactory manner the relative severity of storms and it is probable that this storm is second only to the great storm of 1869. Yet the rivers rose only moderately as a result of this unusually severe storm when it would have been reasonable to expect that a disastrous flood would result from a rainfall which exceeded 12 in. along the Connecticut-Rhode Island state line, and when the area receiving more than 9 in. of rain was 11 per cent. greater than that covered by the greatest storm previously recorded in New England. The area receiving rainfall in excess of 6 in. was 23 per cent. greater than that for the 1927 storm and only 23 per cent. less than that for the 1869 storm, the greatest storm known.

The period of concentration also was unusually short indicating an exceedingly rapid rate of rainfall. This is another reason why a severe flood might have been expected. Yet when the records of run-off are compiled, as shown in Table 6, it is found that the flood was a miserable failure.

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*District Engineer, U. S. Geological Survey.

Only on one stream coming under our observation, the Penobscot River at West Enfield, Me., where the average rainfall over the drainage basin was 100 per cent. greater during the 1932 storm than it was for the 1927 storm, did the peak flow exceed (by 16 per cent.) the peak flow resulting from the November, 1927 storm. This drainage area was located in a region little affected by the 1929–1932 drought. The average rainfall during the 1932 storm was unusually heavy considering the size of the catchment area, averaging 5.64 in. over 6 600 sq. miles. At all other observed points the maximum flow per square mile following the storm of November, 1927 exceeded that for the 1932 storm, in several cases by many hundred per cent. notwithstanding the fact that the average rainfall on the drainage basin was less in the majority of cases during the 1927 storm than it was for the 1932 storm.

Table 6 was prepared for the purpose of showing the remarkable difference in the percentages of the total precipitation falling on the drainage area collected in the river channels during the rises in the river resulting from the storms of 1927 and 1932. Without exception the percentage collected following the 1927 storm was much greater, in some cases many times greater, than the percentage collected following the storm five years later. These percentages collected varied from 14.4 to 87 per cent. with an average of 49 per cent. for the earlier storm, and from 5.5 to 35 per cent. and averaging 15 per cent for the later storm..

There are many factors that affect the run-off resulting from a storm, but it is believed that the difference in the condition of the ground with respect to surface and ground storage probably caused the great difference in the percentage of the rainfall collected in the rivers in the two storms under consideration. Unfortunately, no data are available which indicate the relative position of the ground-water table just prior to the storms. But since New England had not yet recovered from the effects of one of the most severe droughts in history at the time of the 1932 storm, it is reasonable to suppose that the ground-water table was abnormally low, possibly materially lower than it was just prior to the 1927 storm. This condition would provide storage capacity for a greater percentage of the water percolating through the ground from the surface during the 1932 storm than during the 1927 storm. Rain falling on dry ground is quickly absorbed. It is known that the elevation of the ground-water table rises rapidly during and immediately following a heavy rain. It is not until the ground has become practically saturated with water that the rain water begins to run off rapidly into the streams.

The dryness of the soil and the elevation of the ground-water table at any time depends upon the intensity and distribution of the rainfall over the period of time immediately preceding the occurrence of the storm.

Figure 3 has been prepared in its peculiar form for the purpose of showing graphically the rainfall that had been accumulating at the Clinton,

TABLE NO. 6.

LOCATION.	DRAINAGE AREA SQ. MILES.		YEAR.	MAXIMUM FLOW, CU. FT. PER SECOND.		INCHES.	MILLIONS OF CUBIC-FEET.				PER CENT. COLLECTED.
	Total.	Net (b)		Total.	Per Sq. mile.	Weighted Average Rainfall on Drainage Area.	Total Rainfall on Drainage Area.	Total Discharge 8 days following storm.	Total base flow	Run-off collected from storm.	
Penobscot River at West Enfield, Me.	6 600	4 690	1927	60 800	12.5(a)	2.81	30 600	23 700	8 290	15 410	50
			1932	67 900	14.5	5.64	61 400	20 300	3 590	16 710	27
Kennebec River at Waterville, Me.	4 270	3 030	1927	76 800	26.3(a)	2.81	19 800	22 400	5 180	17 220	87
			1932	65 900	21.7	5.16	36 300	11 300	1 740	9 560	26
Androscoggin River at Lewiston, Me.	2 856	2 014	1927	60 000	29.8	4.32	20 200	14 600	2 420	12 180	60
			1932	34 800	17.2	5.16	24 100	8 210	1 380	6 830	35
Merrimack River at Franklin Jct., N. H.	1 510	985	1927	67 000	68	5.46	12 500	10 800	1 240	9 560	76
			1932	14 800	15	4.74	10 800	2 240	705	1 535	14
Merrimack River at Lawrence, Mass.	4 670	3 930	1927	70 360	17.6(a)	4.94	45 000	21 300	3 180	18 120	40
			1932	15 460	3.9	5.22	47 600	4 300	1 240	3 060	6.4
Souhegan River at Merrimack, N. H.	171	171	1927	6 650	39	4.50	1 790	952	82	870	49
			1932	808	8.1	4.90	1 950	129	14	115	5.9
Blackstone River at Worcester, Mass.	31.5	31.5	1927	790	25	4.70	344	132	17	115	33.5
			1932	410	13	6.00	470	59	8	51	10.9
Quinebaug River at Jewett City, Conn.	712	712	1927	12 500	18	5.87	9 700	3 430	622	2 808	29
			1932	5 300	7.4	7.98	13 200	1 450	235	1 215	9.2
Connecticut River at Sunderland, Mass.	8 000	7 740	1927	165 000	21	5.77	103 600	59 100	6 220	52 880	51
			1932	13 200	1.7	2.36	42 400	5 420	3 040	2 380	5.5
Wading River near Norton, Mass.	42.4	42.4	1927	185	4.4	2.25	222	81.5	48.4	33.1	14.4
			1932	74	1.7	3.8	375	28.4	6.2	22.2	5.9

(a) Reduced by amount of regulation.
(b) Reduced by regulated area.

Mass., rainfall station during the period preceding the storm. This diagram indicates the following features concerning the rainfall preceding the 1927 and 1932 storms:

During the first 60 days preceding the 1927 storm, the rainfall had not materially departed from normal although several heavy rains occurred during this period. During the next 60 days preceding this storm, however, the rainfall accumulated very rapidly and reached an excess over normal

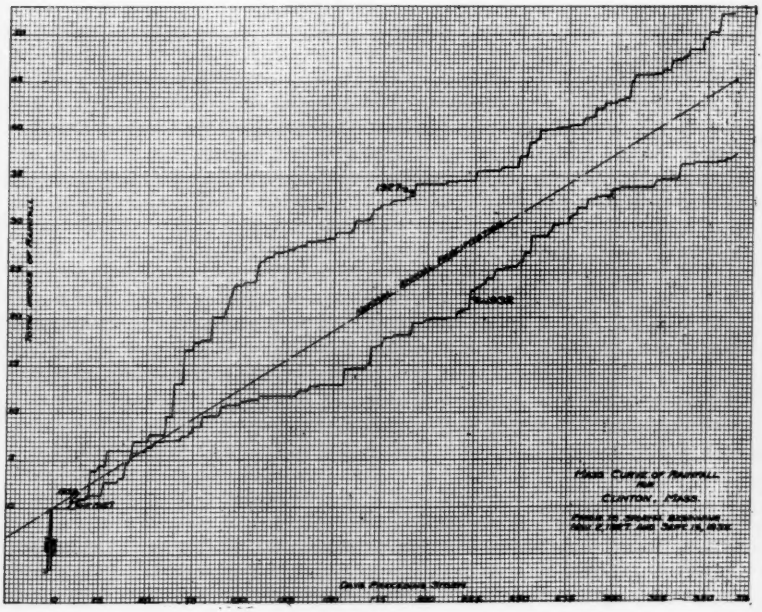

FIG. 3.

of 12.3 in. The precipitation during this period amounted to 21.1 in. or 10.5 in. per month. It also shows that during the first 60-day period preceding the 1932 storm the rainfall was deficient by only half an inch, but that during the period of 60 days earlier this deficiency amounted to a total of over 3 in. This deficiency of 3 in., while not great is in considerable contrast to the excess of 12.3 in. during the same period prior to the 1927 storm.

It appears that in this contrast is to be found the most significant reason for the great difference in percentages of rainfall collected in the rivers following the two storms.

Rainfall run-off formulae, or flood-flow formulas based on rainfall, that do not take into consideration the moisture content of the ground or the relative elevation of the ground-water table may be quite misleading

and of little value except possibly at high elevations where the drainage area is rocky with little soil cover. The condition of the soil cover apparently is the greatest factor affecting the relation between rainfall and run-off. Continuous records concerning the relative elevation of the ground-water table in various sections of the New England States should be collected and if available would undoubtedly greatly facilitate the forecast of low-water flow in the streams and would indicate whether or not conditions were favorable for a damaging flood in case a great storm was experienced.

EXPERIENCES WITH WATER MAIN CLEANING.

BY ARTHUR T. CLARK.*

[*Read September 28, 1932.*]

It is not always possible to obtain new or valuable information whenever a water main is cleaned, but with each cleaning something of interest is sure to develop — some previous idea confirmed or disproved.

Probably all of us have some discouraging operating conditions, but I can look at the pipe shown in Figure 1 and be glad that we have had no such condition. This 16 in. main has suffered corrosion to such an extent that it is in a very bad condition. There remained a clear diameter of but 10¾ in., representing but 34 per cent. of the original area, and the carrying capacity was naturally reduced in proportion.

Corrosion and Tuberculation. Tuberculation may appear in many different forms or shapes and may be due to several causes or combinations of them. Reddick and Linderman may be correct in that tuberculation is due to bacterial growths similar to those producing bog ore or limonite. It may depend upon the amount of hardness or the organic matter in the water, or upon the pH. value. The study of the formation and growth of tubercles is a most important as well as baffling problem and its correction through preventive measures or linings will be extremely valuable in future work.

Today's problems, however, force the operator to deal with waters which, in most cases, are not treated to prevent tuberculation, or to cope with mains with little or no lining in which various degrees of tuberculation already exist. The cost of operating under these conditions is considerable, resulting either in excessive pumping costs or in a decreased flow and pressure.

It is possible, as has been found in some cases, to so treat the water that tuberculation is retarded and existing tubercles are loosened and sometimes partly removed. This, however, usually results in red-water troubles, and the remedy is sometimes worse than the disease. From a chemical standpoint, the hydrogen-ion value at which calcium carbonate equilibrium exists in water, that is at which it neither dissolves nor precipitates, may be determined by the so-called marble test. At this critical point, which varies for different waters, tuberculation is retarded, or in some cases prevented. The short-coming of this treatment is that existing tuberculation is not removed but is hardened. Mechanical cleaning may be resorted to and the main restored to normal capacity quickly, efficiently and reasonably. Cleaning practiced in conjunction with chemical conditioning of the water will hold in check subsequent tuberculation.

*Manager, Greenwich Water Co., Greenwich, Conn.

FIG. 1.— A BADLY CORRODED MAIN IN LAWRENCE, KAN.

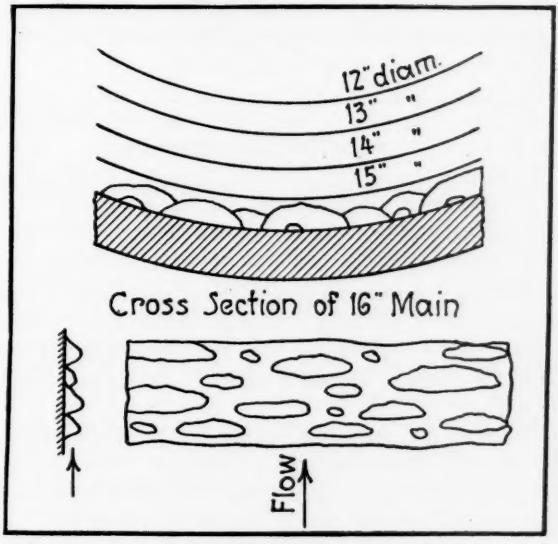

FIG. 2.— SKETCH SHOWING SHAPE, SIZE AND SPACING OF NODULES IN THE POCANTICO, N. Y., MAIN BEFORE CLEANING.

Indicative of the loss of carrying capacity which can be caused by a relatively small amount of tuberculation, were the observations made in connection with a 20-in. main at Lexington, Ky. The tubercles, though evenly distributed and having depths of but $\frac{1}{8}$ to $\frac{3}{16}$ in. resulted in a lowering of the Chezy coefficient of discharge to 90.

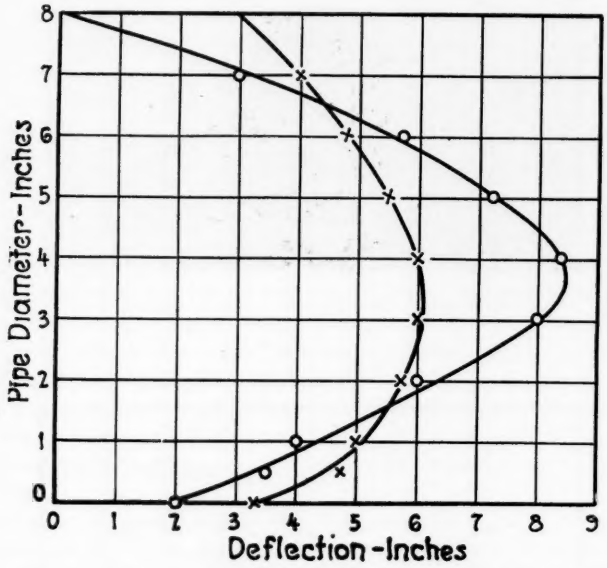

FIG. 3.— PITOMETER DEFLECTION CURVES — TAKEN BEFORE AND AFTER CLEANING AN 8-IN. MAIN.

Figure 2 shows the approximate size and spacing of nodules in the main at Pocantico, N. Y., where a discharge coefficient of 58 was observed before cleaning. It is of interest to note that tuberculation reduced the capacity of this 16-in. main to that of a 12-in. main having a coefficient of 100.

Figure 3 shows a comparison of Pitometer deflection curves for an 8-in. main before and after tuberculation was removed. These tests indicate a pipe coefficient of 0.75 before cleaning and 0.87 after cleaning. (Pipe coefficient equals mean velocity divided by center velocity.) It is also interesting to note that the value of C in the Chezy formula was 63.0 before cleaning and 103.5 after cleaning.

Figure 4 is a photograph (taken from above) of a 24-in corrugated pipe supplied the author by F. C. Scoby of the U. S. Department of Agriculture. It shows the effect of the corrugations on the smoothness of flow of the water and serves to indicate the turbulence created by protrusions such as tubercles on the walls of smooth pipe.

FIG. 4.— WATER FLOWING THROUGH A 24-IN. CORRUGATED PIPE. NOTE THE DISTURBANCE OF FLOW CAUSED BY THE RIDGES OF THE CORRUGATIONS.

Practical Aspects of Cleaning Mains. Where main-cleaning is undertaken the following items should be recorded:

1. Periodic records of pressures and flows — based on (a) accurate levels, (b) calibrated gages or (c) Pitometer or meter measurements.
2. Pipe material and nature of inside protective coating.
3. Age of main.
4. Analysis of water carried.
5. Photograph or sketch showing size and spacing of tubercles — such as is shown in Figure 2.

The cleaning of a water main ordinarily involves excavating, pavement removal and replacement and the opening and closing of the main to be cleaned. In order to defer additional capital expenditure, a seven mile section of a 16-in. main at Pocantico, N. Y., has been cleaned periodically with exceptionally good results, maintaining maximum capacity with a minimum pumping head. After repeated cleanings for several years, the interior surface of the pipe and even the coating remain uninjured. This particular main is located on the Albany Post Road, under a new pavement carrying heavy traffic. Under these conditions a method had to be devised whereby pavement cutting and traffic interruptions could be avoided.

A cleaner pit was constructed at the lower end of the section to be cleaned. It was designed for a 16-in. main under a 4½-ft. cover. This pit makes it possible to open the main with ease, to discharge the cleaning water directly to an adjoining culvert and to remove the pipe cleaner through a standard manhole in the pavement. (See Fig. 5.) The pit is so designed that a section of the pipe can be rolled to one side allowing the necessary cleaning water to discharge into a sump and through an outlet drain. The cleaner upon emerging from the main, tends to stand on end in the sump and is easily removed through the manhole.

Figure 6 shows the method of preparing a standard 12-ft. length of bell-and-spigot pipe with standard Victaulic couplings which was used in the pit on our Pocantico main.

For the insertion of the cleaning machine, "Y" branches have been used successfully at the head of the section cleaned.

Expense of Cleaning. It has often been said that cleaning is expensive — and it is, but in most cases it is not nearly so expensive as the operating cost would be, if cleaning were not resorted to. The saving in pumping-

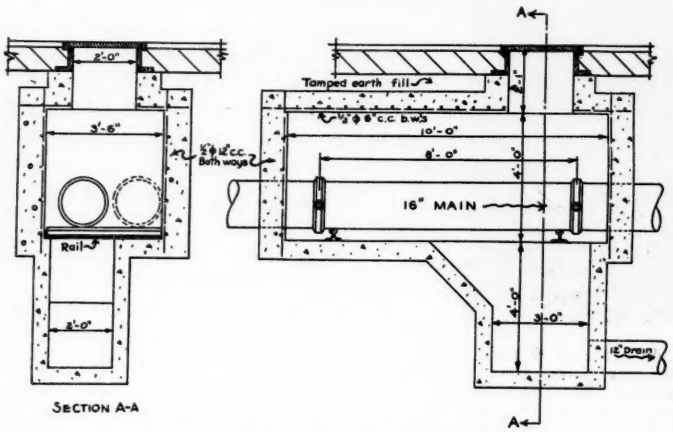

FIG. 5.— DESIGN OF PIPE CLEANING PIT ON 16-IN. MAIN AT DOBBS FERRY, N. Y.

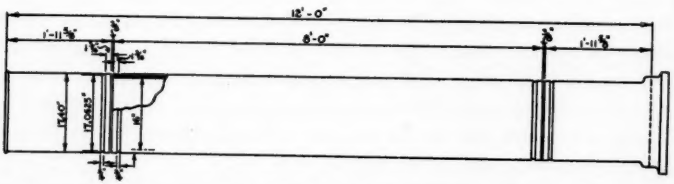

FIG. 6.— METHOD OF PREPARING REMOVABLE SECTION OF PIPE HELD IN PLACE WITH STANDARD VICTAULIC COUPLINGS.

cost alone at the Pocantico plant amounted to roughly $4,800 per year, to say nothing of the value of the increase in the capacity of the main or of the saving through deferred capital expenditure for additional main capacity.

There is another possible item of expense incident to cleaning; that is the length of time that the main is out of service. This can be kept to a minimum through proper and experienced supervision. The possibility of the loss of the cleaning machine resulting in a seemingly futile search is discouraging and expensive, but this can be avoided and long lines can be cleaned safely and with speed. Proper supervision and improved technique

with modern equipment will prevent these costly delays and excavation incidental to the search for a lost machine. Such delays are not only costly but are dangerous. In fact, the length of the shut-off, or the time that the main is out of service depends almost entirely upon the reliability of the operator and the equipment, combined with the best possible coöperation of the local department.

With the arrangements provided at Pocantico, the seven miles of 16-in. main can be cleaned in three hours and the total time that the main is out of service, including tests of the coefficient, averages about five hours.

Results. The main between Pocantico and Dobb's Ferry, N. Y., is cleaned every six months. Figure 7 shows the hydraulic gradient along the

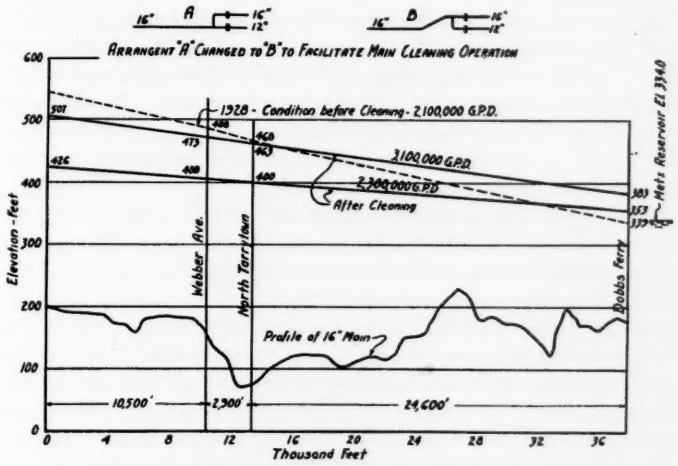

FIG. 7.— PROFILE AND HYDRAULIC GRADE LINE OF 16-IN. MAIN (Dotted Lines Before and Solid Lines After Cleaning).

7-mile section, for different flows, before and after cleaning operations were undertaken periodically. The increased carrying capacity, together with the reduced pumping heads, is indicated. The net saving of roughly $4,800 per year is our reward.

FIRE PROTECTION CLASSIFICATION.

BY GEORGE W. BOOTH.*

[*Read April 26, 1933.*]

At Baltimore, in 1904, a fire spread across a street and from this point continued its onward path of destruction until a large portion of the city was in ruins. Other conflagrations have wiped away millions of dollars worth of property. Their effects upon the fire-loss record may be compared with those of epidemics of disease upon the death rate. The causes of epidemics have been determined and remedies successfully applied. Why should not conflagration conditions be susceptible to corresponding treatment? That was the question which was presented to the National Board of Fire Underwriters at its annual meeting in 1904; the outcome of this discussion was the organization, under the supervision of the Committee of Twenty of the Board, of an engineering corps to look into all of the factors which might be considered as potential or probable elements in the conflagration hazard of American cities.

This engineering corps, after an extensive study of past conflagrations, was instructed to review conditions in the larger cities of the country and to present suggestions for improvements to fire-protection facilities and the elimination or safeguarding of fire hazards.

It was obvious that one of the fundamental needs of every city was an adequate and reliable water supply, as it was evident that the heat of burning buildings could be counteracted only through the cooling effect of large quantities of water. But there must be equipment for effectively applying this water to the burning buildings and the necessary manpower to provide for its handling. It was, therefore, obvious that the organization, equipment and manning of fire departments must be given consideration in estimating the probability factor of the conflagration hazard of cities.

Likewise, there was the question of outside aid which might be called in and the provisions for notifying the fire department of the location of a fire. In these connections, consideration had to be given to all means of transmitting fire alarms, such as the fire alarm system, and the use of telephones. The condition of streets, traffic congestion, and the topography of the city are factors influencing the response of fire apparatus.

In all serious fires the coöperation rendered by the police force is a factor, and the general police duty of enforcing laws, including those in regard to arson and the construction of buildings, is an element deserving consideration.

Generally speaking, every conflagration starts in an individual building, and the probability of such a fire reaching the stage where it becomes

*Chief Engineer, National Board of Fire Underwriters, New York, N. Y.

a conflagration breeder is dependent upon the construction of the building, its size and height, its proximity to other structures, the width of adjoining streets, its class of occupancy, and the probability of a hazardous process or careless housekeeping being the initial start of a fire.

The continued study of conditions in the cities of the United States, of their protective features, and of the potential hazard in the high-value districts, soon indicated that the desirability of some scale or measuring stick to determine in a relative manner the influence which these various factors had upon the fire danger of the various cities. The outcome of this idea was the "Schedule for Grading Cities and Towns with Reference to their Fire Defenses and Physical Conditions" which was adopted by the National Board of Fire Underwriters in 1916, after about two years spent in preparation. The Schedule is based upon the establishment of standards for the various items involved, with points of deficiency depending upon the extent of departure from those standards, which were drawn largely from established practice in those cities where the best service existed; in that sense therefore the members of the New England Water Works Association and their brethren in other cities throughout the country contributed largely to the preparation of the Schedule.

The general principles of the Schedule are not new; they had previously been applied to the grading of dairies and have since been used in the field of sanitation and that of general community health.

The first step in the determination of the relative weight or value of the different items was to list the main factors and to assign a relative value to these principal items. These were grouped under seven principal headings and to each of these a relative percentage value was given.

These headings with their percentage values were Water Supply, 34 per cent.; Fire Department, 30; Fire Alarm, 11; Police, 1; Building Laws, 4; Laws and conditions applying to hazard, 6; and Structural Conditions in the principal mercantile district, 14. These seven principal items may be said to be common to all communities having protection, but there are certain other factors which are not common but which materially affect the general fire hazard and often add largely to the conflagration probability. So to provide for these features, a measure beyond the 100 per cent. has been provided to include extremes in climatic conditions and the unusual happenings of Nature such as earthquakes, tornadoes, hurricanes, cyclones, blizzards and floods.

Having determined the per cent. of value to be assigned to each of these main items in classifying the potential and probable fire hazard of a community, it then became necessary to split these individual larger items into various component parts and determine the relative value of those factors which might influence fire protection or the conflagration hazard.

The adequacy of a water supply is not the only feature which must be considered. There are numerous conditions where the partial or total interruption of this supply may take place, and such an interruption during

a conflagration would be disastrous; or if a small fire occurred at the time of an interruption, it might readily extend to involve a large portion of the city. The determination of the relative value of all those factors in a water supply, which might be considered as limiting the adequacy or as introducing an element of unreliability, was not an easy matter and could not be worked upon any theoretical basis. The work had to be accomplished by a method of trial. Before the final adoption of the scale of values of relative deficiencies, copies of the classification schedule were submitted to various persons for comments and criticisms, including a special committee of the New England Water Works Association. That committee concurred in the general scheme under which a water supply was broken up into its component parts and gave its approval to the relative values assigned to the 32 different items under which the water supply of a city is considered. These items range from the method of employment and the efficiency of the employees down through various items of adequacy not only of supply mains, pumping stations and distribution systems, but also of number of hydrants and the size of hydrants to permit the free delivery of the quantities of water necessary for fire protection. They included reliability features of the source of supply, reserve pumping equipment and fuel and electric lines, the danger of the destruction of the pumping station, the method of installing mains, and the general question of the reliability of these and other mains as influenced by their location, the ease of making repairs, and the spacing of gate valves. In all of these considerations of the water supply due recognition is given to the potential value of a large storage of water and to the closeness of this storage to the point of greatest demand.

A study of the conditions existing in the cities and towns of the country indicates that this question of storage of water in proximity to the communities served is probably the largest factor in its influence on adequacy and reliability of any which go to make up a water system. The value of this storage is considered so great that where it is of such size as to permit domestic supply to be continued to a city for a period of five days it offsets all elements of unreliability except that of the possible destruction of a pumping station by fire, or the situation where a supply main is laid under conditions where repairs would be exceedingly difficult, or in connection with pumping equipment in exceedingly poor condition, or a source of supply where an accident might cause an interruption of exceedingly long duration.

There is not time for a detailed discussion of the various other factors which affect protection or increase the probability of conflagrations, and these will be only briefly touched upon. The analysis of the fire department includes a total of 34 items. These involve many factors of adequacy such as the number of companies, the proper manning, the amount of apparatus, the capacity of fire engines, the amount of hose, the various powerful stream appliances, and other items measuring the amount of equipment neces-

sary for proper fire fighting. To complete the analysis, the method of using the equipment, that is the fire-fighting methods employed, must necessarily be a factor of considerable moment. In this connection there must be a suitable grading of the drills and training, the discipline, the qualification of officers, and other factors influencing the morale of the organization.

The value of a fire-alarm system, including such use as is made of the public telephone, is subdivided into 24 items which include such features of adequacy as the number and distribution of boxes, the amount of equipment at headquarters and at the various fire stations, and also those features which influence reliability, such as the condition of circuits, the frequency of tests and the type of boxes.

The function of the police in connection with fire service is subdivided into four items, and in like manner the influence which building laws have upon the fire safety of buildings is divided into four principal items. But in considering building laws and also in considering the various fire hazards incident to modern business, credit can not be given to the suitability of mandatory requirements without considering whether they are enforced. For that reason the principal value which any city received from the enactment of building codes and fire-prevention ordinances is in proportion to the amount of enforcement carried on by the proper officials.

An outstanding factor in any classification of a city is that introduced by the structural conditions, as these furnish the potential elements with which the features of adequacy and reliability of fire protection must be combined to determine the measure of the city as a whole. These potential factors include the width of streets, the accessibility of blocks, the congestion of a district, the height and areas of the individual buildings, the class of construction, and the protection against spreading fires, both within the building and from exterior sources.

It is interesting to note that in the larger cities of the country, and in fact in practically all of over 25,000 population, the water supply grades materially better than almost any of the other principal factors used in classifying a city. In the smaller communities this is not so likely to be the case. That there should be this difference can be readily understood when it is recognized that a water system is of a dual character. Its first function is that of furnishing an ample and dependable supply of water for the use of the citizens. Therefore, as the city grows in size and the domestic demand increases the time ultimately arrives when the fire demand both as to adequacy and reliability is overshadowed by the demand for domestic consumption.

From time to time new factors arise in connection with a water supply which must receive consideration in applying a classification schedule. One of these factors is before us at the present time, and it would be a great aid if we could receive the opinion of the New England Water Works Association as to the proper treatment of this subject, which is, briefly

stated, the value of steam equipment as a reserve where pumping is by electric or other power. In recent times, the question has been before a large number of cities, mainly on account of the question of economy of operation and also because many of the power companies, to offset the losses due to the depression, are offering service to cities to get them to use the surplus power now available.

The usual procedure is to install sufficient electric pumps to take care of maximum domestic demand with enough capacity to also meet the fire demand but with little reserve in number of units and sometimes with only a single power lead to the station. Under such conditions, reliability of supply as it might be affected by a broken power line, by sleet storms, interruption of service at the generating station, or the need of repair to a pump or motor, are claimed to be taken care of by holding the old steam equipment in reserve. The question then arises as to how long a steam plant can be left idle and still be considered as in serviceable condition; as to how quickly a steam plant can be put into service; as to whether it is reasonable to assume that an operating crew can be assembled; as to whether steam lines, pump packings, and other equipment will be useable after lying idle, and as to whether a steam plant can be considered as being properly tested when it is operated for a few minutes once a year or every six months; or is it necessary to require a 24-hour operation every 30 days, and to insist upon the employment of a sufficient number of engineers and firemen to be able to put this reserve steam equipment in service.

In closing, may I revert to the fact that in most cities the water supply is proportionately better than the other factors involved in fire-protection classification. This is particularly true in New England due to some extent to the opportunities for providing gravity supplies or at least an elevated distributing reservoir of good capacity. Furthermore, the water supplies in this section of the country have been in existence over a span of years longer than in other sections and have naturally reached a higher stage of development in respect to adequacy and reliability. Last, but not least, the influence of this Association in promoting the acceptance of high standards has been for many years a strong factor.

The result of the high standards attained in the water supplies of many New England cities is that comparatively little improvement in classification may be expected on that score. Betterments in fire-department operations, in fire-alarm systems, and in ordinances and their enforcement having to do with building construction and with control of fire hazards, are much more fertile fields for effort.

DISINFECTION OF WATER MAINS.

A ROUND TABLE DISCUSSION OF APRIL 26, 1933.

WARREN A. GENTNER.* The practice of chlorinating or disinfecting newly installed water mains was made a part of the regular main-pipe-laying procedure in Hartford in 1929. Two methods of disinfecting mains were tried out and are in use at the present time. The more commonly used method consists of distributing a hypochlorite powder in the main pipe as it is being laid; this powder reacts with the water, liberating free chlorine.

In Hartford we use a commercial product known as H. T. H., which is put out by the Matheson Alkali Works, and we place about half a pound of this powder in every sixth or eighth length of pipe put into the ground. This is equivalent to about half a pound per 100 feet of pipe laid. The same amount is used regardless of the size of the pipe. This gives a varying dosage strength. In a 6-in. pipe there are liberated something like 18 p.p.m. of free chlorine; in a 16-in. pipe only 3 p.p.m. are obtained. We have considered the saving which might be effected by making the dosage constant and varying the amount of powder that is put in each pipe according to the size of the pipe, but it seems to us that the advantage of having a standard procedure for the field men to follow more than outweighs the slight saving which might be gained by varying the amount of powder placed in the pipes.

As soon as the pipe is laid, water is turned slowly into the section — slowly to prevent the powder from being washed to one end before it has time to react with the water — and as soon as the pipe is filled it is shut off and isolated from the rest of the distributing system and allowed to stand for half an hour or more, after which the main is thoroughly flushed out and placed into regular service. The advantages of this particular type of disinfecting lie in the convenience of its application and the low cost, as well as its effectiveness.

We use this method almost exclusively at present except in cases where a long period of time may elapse between the laying of the pipe and the filling of the section with water. Then there is a possibility of the powder absorbing some of the moisture in the air, if it is left in the pipe, and attacking the lining of the pipe. In such cases, therefore, we resort to the other method of disinfecting, which consists of injecting chlorine gas directly into the section of the main which has previously been filled with water. For this method of chlorination we use a portable chlorinator which is put out by the Wallace & Tiernan Company.

The chief objections to this method of chlorination are the extreme

*Assistant Engineer, Hartford Metropolitan District Commission.

caution necessary in the handling of the chlorine; the time and care required to set up the apparatus and connect it so that there shall be no gas leaks; the necessity of tapping the main to apply the gas; the necessity of keeping the pressure in the main below 25 lb. per sq. in.— preferably 15 lb. per sq. in.— for proper operation of the chlorinator, the necessity of trucking the equipment to and from the job; and, if the work is done in cold weather, the necessity for housing the equipment to prevent freezing.

In operation, the gas is introduced at one end of the main; a flow of water is maintained through the pipe, and test samples of water are taken from the other end of the main. When the orthotolidin test shows the presence of free chlorine, treatment is stopped and the main is isolated from the rest of the distribution system and allowed to stand for half an hour; then it is put into service.

Both of these methods of disinfecting mains require thorough flushing of the main prior to putting it into service; that is a very important feature of this work. We flush our mains wherever possible in two directions to free all possible pockets where the chlorine solution might be found. All mains are flushed through blow-offs into the sewer or through service blow-offs, and sometimes from hydrants. In cases where the extension which we are chlorinating is dead ended and there is no provision for flushing, we insert a 1½-in. corporation cock and flush through a fire hose connected to the corporation cock. The flushing of the main is continued until the orthotolidin test shows absolutely no indication of free chlorine.

EDWARD E. MINOR.* During the past four years we have laid, at New Haven, something like 200 000 ft. of large mains, running from 20 up to 48 in. in diameter. During the past year we brought one line, 36 in. in diameter, right through the center of the city. I had some question as to how much pollution we would find in these large lines if they were not treated, so that we had our laboratory make tests, and we found it was absolutely necessary to sterilize these large pipes. We developed a way of doing that which proved very satisfactory. We did all of this work without the slightest trouble resulting from it, and I have asked Mr. Wellington, who had charge of the sterilizing of the mains, to describe briefly the work.

MARSHALL S. WELLINGTON.† Four and a quarter miles of 48-in. pipe were laid with more than reasonable care. That is, the inside of the pipe was brushed out and then washed with a strong solution of hypoclorite of lime just before being laid in the ditch and put into place. The line was then filled and blown off through a 6-in. blow-off opened about half way for a day. The head on the pipe at this place was over a hundred feet. Samples were then taken for bacteriological tests. These tests showed a count of 500 per cc. at 37° C. with gas being formed in twenty-four hours in four of five 10-cc. samples. On confirming these tubes no bacteria of the *Coli-Aerogenes* group were found. After two more days of blowing off, samples showed the

*Engineer and General Manager, New Haven Water Company, New Haven, Conn.
†Superintendent of Chlorination, New Haven Water Company, New Haven, Conn.

same count. While no bacteria of the *Coli-Aerogenes* group were found, it was thought best to sterilize this main as the counts indicated that it was not as free from contamination as could be desired; particularly as the water entering the main had a count at 37° C. of 15 colonies and no gas showed in 48 hours in five 10-cc. samples.

It was with some misgivings that chlorine was added at the beginning of the main, due to the long distance that it would have to travel to reach the blow-off and to the physical difficulties of adding the chlorine. Fortune smiled upon us, as we were able to obtain a residual chlorine at the blow-off of 1.0 p.p.m. This, after standing in the main over night, left a residual of 0.6 p.p.m. and after standing five days a residual of 0.3 p.p.m. Samples taken from the same blow-off at this time gave a count of 10 bacteria per cc. at 37° C. with no gas showing in five 10-cc. samples.

Success being obtained at this point, it was decided to try 2 500 ft. of 42-in. pipe and 2 000 ft. of 36-in. pipe that had now been completed on the end of the 48-in. pipe. The majority of the 36-in. pipe was a crossing under the Quinnipiac River. Before sterilizing this piece of pipe the counts were approximately the same as in the 48-in. pipe before sterilization. A residual chlorine of 0.5 p.p.m. was obtained at the end of the 42-in. pipe and one of 0.03 p.p.m. at the end of the 36-in. pipe. These amounts were considerably lower than desired, but samples taken at the points showed a bacteria count of 40 per cc. at 37° C. with no gas showing in five 10-cc. samples. This indicated that the water in the main was safe to use.

The next piece of main was 36 in. in size and extended for some 2 750 ft. When this was filled and blown off for a day, samples taken showed a count of over 500 bacteria per c.c. at 37° C. with gas forming in all five 10-cc. samples in 24 hours. Two of these gas-forming tubes confirmed for the *Coli-Aerogenes* group. Chlorine was added to one end of this piece of pipe while it was being blown off from the other. Ten pounds of chlorine were added before a residual of 10 p.p.m. was obtained at the other end. The line was allowed to stand over night, then it was blown free of chlorine and further samples were taken which showed a count of 30 per cc. at 37° C. and no gas in 48 hours in five 10-cc. samples.

The next section of pipe of the same diameter was 2 000 ft. long. It showed approximately the same counts as the previous section. This was sterilized in the same manner with the same results. From the results up to this point it was decided that all of the main to be laid should be sterilized without making bacteria tests first. The principal reason for this was that time was a factor and we did not want to wait 48 hours to find that the samples were bad and delay another three days until the main was sterilized and tested.

Two sections of the total pipe were laid under the worst possible conditions, but upon the addition of chlorine to give a residual chlorine of 20 p.p.m. and being allowed to stand for 48 hours gave bacteria tests as good as any other section of the pipe. To make sure that the known pollu-

tion had been removed from these sections, they were allowed to stand another 48 hours when more bacteria tests were made. These tests showed no increase in the bacteria.

One of these sections was flooded with sewage from a broken sewer, the other was laid just below high-water mark across a marsh some two hundred feet below a fat-rendering plant. At every tide all of this section was filled with this highly polluted water. After the addition of chlorine, the residual being 20 p.p.m. at blow-off, the bacteria counts were under 30 per cc. at 37° C. with no gas in 48 hours in five 10-cc. samples.

Every length of pipe laid, some 53 600 ft., was cleaned out and washed with a strong solution of hypochlorite of lime before being laid.

Just a few words about the method of using chlorine to sterilize these pipes. For the long lengths of 48-, 42- and 36-in. pipe a Wallace & Tiernan Co. emergency direct feed panel with an injector was used. The water pressure to operate this injector was brought in from a stone quarry which operated from eight in the morning until four in the afternoon. This limited the time of using the chlorine apparatus to these hours. Under these conditions it required three days to carry the chlorine through the five miles of pipe. This method of sterilizing such a long length of big pipe is not recommended. It is much better to take it in shorter sections between gates as was done on the rest of the pipe.

For the rest of the mains, the injector panel only was used as the water pressures were such that the controlling feature was the capacity of the injector. The injector panel was set up at the gate between two sections and a cylinder of chlorine standing on platform scales attached directly to it. The water connection to injector panel was taken, in most cases from an air cock on one side of the gate. If such a fitting was not available a nearby hydrant was used. The chlorine solution was added through an air cock on the other side of the gate. The by-pass around the big gate was opened about two and a half turns according to the location of the blow-off. The blow-off was either a regular blow-off into a sewer, stream or a hydrant.

As the pressure in the main in most cases was between 30 and 35 lb. per sq. in., and the difference in pressure to operate the injector was three to one, it required very close control of the gates to make it possible to operate the injector. When the injector was working right the amount of gas and the amount of water going into the main were properly proportioned. This condition would not be encountered very often. In most cases the actuating pressure for the injector would be large enough so that the main, being filled, could carry a higher pressure and thus have a much higher rate of flow. In these cases a means of measuring the amount of chlorine being fed would be needed. Using chlorine at six cents a pound, the cost for sterilizing the first long section was $8.34. Chlorinating the remaining 36-in. pipe cost $7.68. The total amount of chlorine used was 267 lb.

It is well to mention here that 5 000 ft. of 20-in. pipe leading to a 100-

000-gal. standpipe and 2 500 ft. of 24-in. pipe from the standpipe were also sterilized. This was done very easily as the chlorine apparatus and pump were ready to run when the standpipe was completed. The pump was started and the chlorine set at a rate of 40 lb. per day. In eighteen hours the stand pipe was full as well as the lines. The residual chlorine at the end of 24-in. line and in the standpipe was 3.0 p.p.m. Three days later the standpipe was drawn down to check some valves. A sample taken at this time showed a bacteria count of 23 per cc. at 37° C. with no gas showing in 48 hours in five 10-cc. samples. Whether this line and standpipe needed to be sterilized, I do not know. They were only filled with highly chlorinated water which made samples of their true quality undeterminable. With the experience of the other lines in mind, it is fairly safe to assume that they did need to be sterilized.

Any accurate estimate of the cost of sterilizing mains is rather difficult. As a general basis, I will say that if the chlorine used costs six cents a pound, it will cost thirty cents to disinfect 1 000 ft. of pipe. In our work one man was needed at the apparatus at all times to keep the innocent bystander away. Half an hour was consumed to set up the apparatus and take it down. I visited the work two or three times while it was being performed to see that the equipment was working satisfactorily and to test for residual chlorine. It took about six hours to get the proper amount of chlorine into the water.

On account of the low cost, the ease with which it can be done, and the safety that it gives to the consumers, it is my opinion that all mains that cannot be blown off with a sufficient quantity of water to give a strong scouring action in the main should be sterilized with chlorine.

MR. MINOR. I think that one of the values which discussions of this kind have for us all is to compare different ways of doing the same work. Mr. Wellington has told about the chlorination we depended on for sterilizing the pipe, but as the pipes were laid we used H. T. H., only instead of putting it in the pipes and letting water flow through them, we made a very strong solution, and as the pipes were laid we had a man crawl into the pipe with a flit gun and spray the surface with the solution. Our feeling was that by spraying this strong solution on the sides of the pipe we were putting it right where it was needed; whereas the powder which was put on or near the bottom might possibly be washed along and certain sections of the pipe might not be covered. I recall that when we started laying this large pipe line, I was horrified by someone telling me that they found feces in the pipe. That seemed to me pretty bad. I told the foreman, and he said he had wiped the pipe out very carefully and it was perfectly all right. However, we developed the routine of having H. T. H. on the job, and as the pipe was laid you could see a man go in with the flit gun and spray the disinfecting solution over the surface of the pipe. But we did not depend on that; we depended on the chlorination, which, as Mr. Wellington said, worked out very successfully and gave us what we thought was a perfectly sanitary pipe when we finished.

USE OF UNEMPLOYED AND RELIEF UNDERTAKINGS ON WATER WORKS.

A ROUND TABLE DISCUSSION OF APRIL 26, 1933.

HARRISON P. EDDY.* It is common knowledge that expenditures for relief of destitution resulting from unemployment have reached unprecedented amounts. For example, the city of Worcester in 1932 expended $1 802 510.19, an average of about $8 per week for each "case," a case being an individual or a family, regardless of number of dependents. This amount excludes the cost of administration and some clerical service and was used for "outside relief," *i. e.* it does not include old age and soldiers' relief payments. The annual expenditure is equivalent to a payment of $9.23 for each of the 195 311 inhabitants (1930 census). The total amount of taxes for 1932 was equivalent to $66.30 per inhabitant. This figure includes taxes on real estate, personal property, corporation taxes, income taxes — all taxes paid by the citizens of the city of Worcester, together with $50 000 contributed voluntarily by the city employees. The cost of outside relief, therefore, was equivalent to 13.9 per cent. of the total amount of the taxes. If the debt service charges were deducted the percentage of the net available taxes for running the city would be 17.63.

In certain communities there has been an effort to require recipients of unemployment relief to perform work in return. Often the work has been of little or no value, largely because of lack of adequate planning, management and supervision. More recently there seems to have been a decided development of public opinion in favor of utilizing this potential supply of labor to secure needed permanent public improvements, such as water-works betterments. There are several difficulties in the way, however. Some of the recipients are not willing to work, but it is believed the proportion is now small, although probably it is increasing and will continue to grow as long as a virtual dole is countenanced.

Some permanent improvements which, under ordinary conditions, would be classed as needed, are not urgent necessities at the present moment because of the conditions caused by the depression. This is peculiarly true of water-works betterments. Since the depression began there has been a gradual decrease in water consumption, until now in some cities consumption is as low as it was fifteen or twenty years ago. To proceed with improvements under such conditions requires reasonable optimism, faith in the future and in the municipality, and a willingness to invest for the future. In some cases this would mean that less revenue from the water works could be utilized for other public enterprises and for reducing the tax levies; in other cases, actual contributions from tax receipts would have

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to be made to aid in financing the betterments. Both courses are unpopular.

Perhaps the most serious obstacle to undertaking betterments is the necessity for providing tools, supplies and materials in order to render the labor useful. This applies with exceptional emphasis to many water-works betterments. For example, in Concord, Mass., in the laying of certain cast-iron water mains only 36 per cent. of the cost went for job labor — that is, the labor actually on the job, which was welfare labor. On the other hand, at Bristol, Conn., 62½ per cent. of the cost of raising a dam went for labor. On certain sewer construction Mr. Frank A. Barbour, a member of this Association, was able to devote 64 per cent. of the cost to payment of job labor. Some officials hesitate to spend $1.50 to $3 for one dollar's worth of labor. Many city officials approach the problem by saying: "We have only to spend a dollar to support the man; why spend 50 cents to $2 more to supply tools and materials for the sake of getting some permanent work in return for the payment?" It is pertinent, however, to point out that local merchants often benefit from the sale of tools, supplies and materials to the water works. Also, if all labor, wherever employed, is considered, a large proportion of the expenditure, perhaps 75 per cent. or more, goes to compensate labor.

There are three major reasons for furnishing relief by employing men on needed permanent public works:

First, to maintain the man's self respect, individuality and independence;

Second, to create buying power, which is necessary to stem the tide of deflation and establish an upward beneficent spiral which will lead to prosperity; and

Third, to secure works of real value in return for the vast sums which must be expended for relief of the unemployed.

The merits of the first — *i. e.*, the maintenance of the man's self respect — are conceded by all and need not be discussed. Perhaps that is the most important of the three points. The desirability of creating buying power is conceded, but many believe that it is impossible for any practical public-works program to achieve a substantial measure of success because of the doubtful financial soundness of some municipalities and the inability to collect higher taxes or even taxes at present rates. Natural thrift leads to favorable consideration of means for securing needed permanent public works from the moneys expended for outside relief. This third reason for employing men on such work probably has the greatest appeal for local voters.

It seems evident that those municipalities which are in sound financial condition should make every effort to build needed public works with labor which otherwise would be unemployed. If such employment is to be of material assistance in stemming the tide of deflation it must be general over the entire country and the movement should go forward everywhere at one time. This condition can be brought about only through the in-

itiative and financial assistance of the federal government. It is this consideration which has led to the plan for federal grants to states and their political subdivisions.

Various proportions of federal and local contributions have been suggested; that which appears to be most favorably considered is 50 per cent. by each party. The sum which should be provided by the federal government also is variously estimated, some placing it as high as six billion dollars or more. If the sum was fixed at one billion dollars and allocated among the states in proportion to their populations by the census of 1930, the several New England states would receive the following amounts:

Maine	$6 500 000
Vermont	2 900 000
New Hampshire	3 800 000
Massachusetts	34 600 000
Rhode Island	5 600 000
Connecticut	13 100 000

If the federal grants were fixed at one billion dollars and the local contributions at an equal amount, and the proportion of job labor is assumed at 50 per cent. of the total cost, money would be made available for 1 736 000 men on job labor for one year of 240 days at 40 cents per hour, six hours per day; or 2 600 000 men employed anywhere if 75 per cent. of the money were available for labor.

A number of estimates have been made of the work available for such employment. A recent compilation for Massachusetts, excluding all local street improvements and buildings of various kinds, aggregates $56 000 000, of which $7 605 000 is for water works. This sum is made up of work which already had been subjected to engineering study. Doubtless the list could be increased to double this amount.

This compilation comprises $33 462 000 or 59 per cent. for municipal projects and $23 179 000 or 41 per cent. for state projects. If 59 per cent. of the Massachusetts quota, or $20 400 000, were to be allocated to the municipalities in proportion to their populations the amounts to be received may be illustrated by the following:

Worcester's	quota — 4.59 per cent.,	or $937 000
Lowell's	quota — 2.35 per cent.,	or $480 000
Quincy's	quota — 1.69 per cent.,	or $345 000
Newton's	quota — 1.54 per cent.,	or $314 000

If it is assumed that the cost of each project is to be divided equally between the federal grant and the proceeds of a municipal bond issue, would it be possible to float the necessary bonds?

Although there may be some municipalities which could not and should not issue bonds for construction of any kind, there are many cities and towns which are in reasonably sound condition. Notwithstanding this fact, there is at the present time hesitation on the part of investors to buy

municipal bonds. Unfortunately this is based in part at least upon popular disparagement of all municipal management.

It is often said or implied that the present difficult financial position of municipalities is due to mismanagement, extravagance and graft. It has been said that municipalities have been "spending money like drunken sailors."

I wish to take vigorous exception to these implications. The trouble is not generally due to municipal extravagance. The primary cause is the widespread depression in business which has thrown vast numbers of unemployed onto the municipalities for support and so curtailed the incomes of others that they have not been able to pay their taxes. Based on my observations and experience, it is my judgment that, in general, municipal officials compare favorably with those of private business in honesty and efficiency of management.

In conclusion, it would seem that there is a probability that the federal government will embark — rightly or wrongly — upon this plan, and an effort will be made to put as much work as possible under construction before next winter. Therefore, every water-works man should be prepared to suggest what betterments are most necessary and advisable, selecting in so far as practicable those which will provide the largest proportion of job labor, in order to make a strong appeal to the local authorities.

W. A. MacKenzie.* It is evident that the public cannot and should not rely upon the Federal Government to carry out all of the constructive measures necessary for a revival of confidence and buying power among the people. Every company, municipality and individual, should coöperate in creating work so that our business structure may remain intact.

The large federal projects, some of which are under way, appear to be concentrated in a few sections of the country. These efforts are praiseworthy, in so far as their purpose is the restoration of confidence and employment, but this is a task which every town, borough, and city is capable of sharing. The construction of sewers and the laying of water mains is one of the best methods of putting the unemployed to work. Assessments for sewer construction should be delayed until some future date. As for water mains, additional revenue, adjusted rates and lower insurance rates tend to make them self-liquidating structures. These attempts to relieve the labor situation are commendable since the work provided enables property owners to pay their water rents and assist in paying their taxes. This prevents the placing of tax- and water-liens on property, thus decreasing the possibility of mortgage holders foreclosing.

The present low cost of materials offsets somewhat the slightly higher costs for doing the work with hand labor. The average cost for ditching and laying 8- to 12-in. water mains is 30 to 40 per cent. of the total cost, while sewer construction averages 45 to 60 per cent. of the total cost. The only drawbacks or arguments against municipal work are the inefficiency

*Superintendent, Wallingford Water Department, Wallingford, Conn.

due to the unwillingness of some men to work, and political interference in the placing of jobs with other than property owners, a condition which should be set aside during this period of crisis, and one with which emergency measures as a whole have to contend.

A notable example of vigorous expansion is to be found in the city of Meriden, Conn., which has constructed many miles of sewers and water mains. These are additions which increase materially the value of property and provide a source for future revenue.

The Wallingford Water Department during the years 1931 and 1932 has laid over four miles of water mains from 8 to 14 in. in size. All of the ditching was done by hand labor, and the approximte cost, including filling and laying, was 45 to 75 cents per linear foot, depending, of course, on the ground conditions and size of pipe. About 150 property owners paid water rents by working during 1931, and 300 during the year 1932.

The year 1932 ended with practically 100 per cent. collection of water rents to date. This year the department is preparing to lay a long section of 24-in. main-line pipe and hopes to collect the water rents by furnishing the work. The prevailing wage has been 40 cents per hour, and the best men are allowed to work long enough to receive after paying the water rent, from $4 to $7 in cash. As a rule, the property owners are far better and more efficient laborers than the floaters. Probably 400 property owners will work out their water rents this year. Since we have about 2 300 services, this means that almost one-fifth of our collections are now being made by providing work.

The Wallingford Water Department has also cleared waste brush land on the water shed and planted some 30 000 Red Pine and Norway Spruce trees, two-and three-year transplants. The Norway Spruce is planted between the 8-ft. rows of pines with the idea of cutting them out within a ten-year period for Christmas trees. This will reimburse us for a large part of the expense.

On the basis of this experience, I recommend that all of us as engineers and superintendents, representing various departments and municipalities throughout New England, redouble our efforts along such constructive lines.

I do not think there is any other way to collect water rents from unemployed property owners. I think it would be far better for municipalities to borrow from the Reconstruction Finance Corporation the money to buy the materials or pipe, and instead of giving doles, to pay for labor, collect rents and help collect taxes. There will be a few men who won't accomplish much, but the time they are on the job is short and they can be put off somewhere on the job where they do not interfere with the main job itself.

THEODORE L. BRISTOL.* In Ansonia we have about 3 000 acres of woodland. When the depression first came on about three years ago a lot of men were thrown out of work. Though by no means down and out, they

*President, Ansonia Water Company, Ansonia, Conn.

did not know what to do with themselves. It occurred to us that it would be a good plan to let them cut wood on our land. We advertised the fact that we would allow people to cut wood free of charge. We said that they would have to go to the job themselves and would have to cart the wood. We had about 300 applications the first year (1929–1930). The next year we had 400. This year we have had over 1 000 men who have cut wood, and I presume they have each cut well over a cord. Some take their boys with them, who are able to drag the wood out. We have given them the privilege of cutting all white birch, all defective trees, and what we call wolf trees — big field trees that stood on the old farms when we bought them.

This undertaking has been a great success. Usually the work has been cut off early in the year because of the sap getting into the wood, and has not been begun until late in the fall on account of the fire danger. This year the work has kept right on.

We have two foremen who superintend the cutting. Two gangs of fifteen men at a time are permitted to cut until they have all the wood they want, or ought to have. We do not allow them to sell the wood.

J. J. Watson. About two years ago when relief work was begun it was a problem for the Mayor and the Board of Welfare to find out what they were going to do with surplus labor. The Board of Water Commissioners, therefore, took charge. We started out the first month or so with approximately 50 men a week, and before we got through we were handling 225, and as high as 250 a day. We built highways around our reservoir, cut fire lines through the woodland, and had an arrangement with the Board of Charities whereby, if they supplied the men, we gave them the privilege of cutting dead wood, for a dollar a cord. Unfortunately we have not been able to collect a dollar. The Board of Welfare Administration has changed, and the chairman wrote me a letter a short time ago saying, "Try and forget it; we are not assuming the obligations of our former commissioners." Nevertheless the undertaking has been a good thing and has given us an opportunity to do a lot of work on our watersheds that would have cost a lot of money under ordinary conditions. We have trimmed easily 200 000 trees anywhere from 15 to 18 years old; we have also trimmed trees down to as low an age as five years. We cleaned up our brooks and put them in better sanitary condition and we have a lot of other work which can be done.

We are not the only department that is doing this kind of work. The Department of Public Works has built a great many roads.

The Welfare Department, to begin with, has allowed the men the equivalent of 40 cents an hour in groceries. The Water Department has added to this 50 cents a day.

George F. Merrill.* I would like to ask if the water department pays the workmen's compensation insurance?

Mr. Watson. We have not as yet. Some say we are obligated to do it; others say we are not. Of course, in taking a lot of green men and putting

*Ware, Mass.

them to cutting wood, they will have accidents. We have had anywhere from 130 to 200 men at work, and we have somebody experienced in first aid work with them. But we have had to pay some very expensive doctors' bills.

HUGH MCLEAN.* Some work is being done at Holyoke along these lines, only in a different way. It is handled by one of the agents of the American Legion and Soldiers' Relief Unemployment. He has taken the men out to the water-works forests, and the Water Department has detailed one of its foremen to designate the trees which are to be saved and those which may be cut down. Some 600 cords of wood have been cut. The men draw it out to the highways, and the city trucks draw it to the city farm where it is cut into firewood. We have kept about 50 men at work. They were getting wages from the Soldiers' Relief Fund and are rendering service for it.

*Holyoke, Mass.

THE WAY OUT

BY E. SHERMAN CHASE.*

[*Read March 8, 1933.*]

Never before during this period of stress has there been a more opportune moment to start a movement of nation-wide scope which will lead us out of the ever-increasing fear of economic disaster. The closing of the banks of the whole country may well be the basic action from which concerted efforts will lead us out of the morass of fear into which we have been steadily sinking. There are many economic and social problems confronting us today; problems relating to the war debt, international trade, world peace and internal problems too many to enumerate. For the immediate present, and only for the immediate present, these problems must be relegated to the background. The problem which now overshadows all others is that involved in unemployment on such a tremendous scale that millions in this land are living in direst poverty and millions more are fast approaching the same condition. Every one recognizes the extreme gravity of the situation. Nearly every one proposes a remedy.

Remedies have been tried, one after another, and one after another have failed or at best have but stemmed the tide of economic events for a fleeting instant. On and on has the sinister flood of adversity rolled over the country until it threatens to engulf us all.

The most widely advertised of the remedies is that which involves economy. To a degree this policy is wise in so far as it relates to unproductive spending. On the other hand it has apparently brought about a most disastrous effect upon the minds of the American people, to whom economy means simply *not* spending, and it is this failure to spend which brings about stagnation in industry and commerce. Failure to spend on the part of those who are able to do so is much as though a water superintendent should shut off his reservoir during a summer period of drought, in order to conserve water lest it should never rain again. Wise conservation of resources — yes. Niggardly hoarding — no.

The fundamental reasons for this and every other depression are two ugly characteristics of mankind — greed and fear. Remove the cause and the disease is prevented. The eradication of greed cannot be accomplished in a day or in a century, but fear — its twin evil — can be changed to courage overnight. No process designed to restore economic prosperity can begin to function until the fear which paralyzes and has paralyzed the country can be removed and replaced by optimism.

Now is the time to launch a campaign to restore public confidence and faith. The national government can perform an inestimable service by

*Of Metcalf and Eddy, Consulting Engineers, Boston, Mass.

protecting the integrity of the banking system and of the currency, but the foundation for better times must be built by the people themselves. Never since the world war have the American people been so ready to follow any national unified policy for the public good. The policy which I wish to submit is very simple. Briefly, it is the establishment of a nation-wide organization similar to that of the National Economy League, but working in the individual communities in the manner of the drives for welfare and charity funds. But this organization will not seek to secure funds or pledges of money, but simply obtain pledges to spend at once, from every one who is able to do so. Such expenditures should be for needed commodities or services which have been deferred due to the fear of exhaustion of individual resources. Such pledged expenditures must, of course, be within the means of the individual making the pledge. On the other hand, no individual should withhold a pledge because the amount involved is of necessity small.

Even the most trivial of pledges, if numerous in the aggregate, will produce benefits. For example, let every man pledge to have his hair cut every three weeks instead of once a month. The buying capacity of the barbers throughout the country would be increased very substantially. Let those of us who need new suits of clothes, buy them at once. A whole chain of beneficial results will follow; the increased purchasing power of the clothier, the tailor, the textile worker and eventually of the farmer and of the machinery manufacturer. *For one to prosper, all must prosper.*

For this policy to be successful there must be an organized unity of action. The individuals here or there who believe in this theory can accomplish nothing of their own efforts alone. Results can be produced only by concerted increase in buying on the part of the whole nation. Campaigns to raise relief funds are failing not because people are ungenerous, but because they fear for their economic stability and because they fear that such funds are being poured into a bottomless pit with no limit to its capacity. People fear to spend because they see no end to the depression and no way out. Assure people that all are joining in a movement which must inevitably restore good times, and there will be no hesitation. In the aggregate the American people have unlimited resources and if these can be tapped at once all will benefit promptly. If they are not tapped, disaster will ensue to all. Declare a moratorium on saving for one month, six months or a year; start spending for useful goods and services; opportunities to save will come later and more abundantly. This and this only is the way out.

PROCEEDINGS.

March Meeting.

Hotel Brunswick, Boston, Mass.

Wednesday, March 8, 1933.

President Richard H. Ellis in the Chair.

Secretary Gifford announced the election by the Executive Committee of the following new member:

Percy A. Shaw, Water Works Office, Manchester, N. H.

The President. I shall read a letter from Mr. S. E. Killam, Chairman of the Committee on New Form of "Summary of Statistics."

Mr. Richard H. Ellis, March 7, 1933.
President, New England Water Works Association,
Dear Sir:

Your Committee on new form of "Summary of Statistics" desires to make a report of progress.

We have been surprised at the few in number of those who are responsible for water-works reports who use the "Summary of Statistics" as adopted by this Association in 1902. The reason of this non-use of the form by the active members of the Association is not as yet clear to your Committee, but we do feel that there must be something lacking in this form which is the reason the members have not adopted it generally. For this reason we believe it advisable to give out at the meeting on March 8th copies of the blank form as adopted in 1902 with the request that if the members have any changes in or additions to this form which they wish to have considered by the Committee that they would make note of these changes or additions and return the same to the Association so that the Committee may have the benefit of their suggestions.

(Signed) S. E. Killam.

Appointment of Nominating Committee

On motion of Mr. Charles W. Sherman, duly seconded, it was Voted that the President be authorized to appoint a nominating committee.

A paper "The Way Out," dealing with the need for stimulating the rate of circulation of money was read by E. Sherman Chase.

A paper "Historical Notes on a Line of 10-inch Cast Iron Pipe, laid in 1854, Pennichuck Water Works, Nashua, N. H.," was read by D. C. Calderwood, Engineer, Pennichuck Water Works. Messrs. Rober Spurr Weston and Roger W. Esty took part in the discussion.

A paper "Field Tests on Large Meters," was read by Nelson Boardman, Assistant Superintendent, Danvers Water Works, Danvers, Mass. Messrs. Reeves J. Newsom, Wilfred J. Wetherbee, and Stephen H. Taylor participated in the discussion.

A paper "Some Electrical Kinks in Water Works Practice," was read by J. H. Read, Assistant Superintendent, Manchester, N. H., Water Works. Messrs. Henry T. Gidly, David A. Heffernan, Edward E. Minor, Herbert C. Crowell, William Plattner, E. J. Titcomb, Gordon M. Fair, Wilfred J. Wetherbee, A. N. Tiffany, George F. Merrill, Arthur V. Harrington, Harold W. Griswold, and Edwin A. Taylor took part in the discussion.

(*Adjourned.*)

Special April Meeting.

Hotel Burritt, New Britain, Conn.

Wednesday, April 26, 1933.

Morning Session.

President Richard H. Ellis in the Chair.

The President. Gentlemen, it has been the practice of the Association in the last few years to hold at least one regular meeting outside of Boston, and it gives me great pleasure to open this meeting in New Britain, Conn.

Secretary Gifford announced the election by the Executive Committee of the following new members: J. William Nolan, Water Commissioner, Somerset, Mass.; Frank J. Callahan, Superintendent New Haven Water Co., Hamden, Conn.; Henry Francis Martin, Foreman, Melrose, Mass.; Marshall S. Wellington, Superintendent of chlorination, New Haven Water Co., New Haven, Conn.; Walter J. Connor, Chemist, New Haven Water Co., New Haven, Conn.; Thomas J. Fleming, Superintendent of Public Works, Waterbury, Conn.

The morning session was devoted to round table discussions. The first subject for discussion "Disinfection of Water Mains," was introduced by Warren Gentner, Assistant engineer, Hartford Metropolitan District Commission, and was discussed by Messrs. E. E. Minor, Marshall S. Wellington, Francis H. Kingsbury, F. O. Almquist, E. Sherman Chase, and Robert Spurr Weston.

The next subject discussed was "Shutting-off Water for Non-payment of Water Bills." This subject was introduced by Mr. F. O. Almquist, Assistant Sanitary engineer, Connecticut Department of Health, and was discussed by Samuel H. MacKenzie, D. W. Harford, J. J. Watson, L. G. Carlton, Arthur V. Harrington, and Henry T. Gidley.

The third subject discussed was "Use of Unemployed and Relief Undertakings on Water Works." Mr. Harrison P. Eddy opened the discussion, which was participated in by Messrs. William A. MacKenzie,

Theodore L. Bristol, Robert W. Glasheen, Warren J. Scott, Samuel H. MacKenzie, J. J. Watson, George F. Merrill, and Hugh McLean.

After luncheon President Ellis introduced the Honorable George A. Quigley, Mayor of New Britain, who welcomed the members of the Association to New Britain.

Afternoon Session.

A paper "Fire Protection Classification," was read by George W. Booth, Chief Engineer, National Board of Fire Underwriters, New York City. Warren J. Scott, Linn H. Enslow, and Charles W. Mowry took part in the discussion.

A paper "Cross Connections in Plumbing Fixtures," was read by James R. Walker, Plumbing Inspector, Waterbury, Connecticut. Warren J. Scott, Charles W. Mowry, Robert Spurr Weston, Arthur V. Harrington, George F. Merrill, and Henry T. Gidley participated in the discussion.

(*Adjourned.*)